PB88-113295

Construction Quality Control and Post-Construction Performance Verification for the Gilson Road Hazardous Waste Site Cutoff Wall

Goldberg-Zoino and Associates, Inc.
Newton Upper Falls, MA

Prepared for

Environmental Protection Agency, Cincinnati, OH

Aug 87

U.S. Department of Commerce
National Technical Information Service
NTIS

TECHNICAL REPORT DATA
(Please read Instructions on the reverse before completing)

1. REPORT NO. EPA/600/2-87/065	**2.**	**3. RECIPIENT'S ACCESSION NO.** PB88 113295/AS

4. TITLE AND SUBTITLE Construction Quality Control and Post-construction Performance Verification for the Gibson Road Hazardous Waste Site Cutoff Wall	**5. REPORT DATE** August 1987 **6. PERFORMING ORGANIZATION CODE**
7. AUTHOR(S) Matthew J. Barvenik & John E. Ayres	**8. PERFORMING ORGANIZATION REPORT NO.**
9. PERFORMING ORGANIZATION NAME AND ADDRESS Goldberg-Zoino & Associates, Inc. 320 Needham Street Newton Upper Falls, Massachusetts 02164	**10. PROGRAM ELEMENT NO.** Y105 **11. CONTRACT/GRANT NO.**
12. SPONSORING AGENCY NAME AND ADDRESS USEPA Hazardous Waste Engineering Research Laboratory Cincinnati, Ohio 45268	**13. TYPE OF REPORT AND PERIOD COVERED** Final **14. SPONSORING AGENCY CODE** EPA

15. SUPPLEMENTARY NOTES

Project Officer: Stephen C. James ph. 513-569-7877

16. ABSTRACT

This report describes assessment activities undertaken to evaluate the effectiveness of a soil/bentonite backfilled cutoff wall (slurry trench) installed for the purpose of hazardous waste containment. The work includes development and evaluation of field quality control tests, evaluation of electronic piezocone instrumentation for post-construction verification of backfill homogeneity, and evaluation of cutoff wall bulk hydraulic conductivity via hydraulic stress testing. This information in this report is useful to those involved in the feasibility study, design and/or construction of cutoff walls as a hazardous waste remediation technology.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Slurry Wall Field Verification Superfund		

18. DISTRIBUTION STATEMENT RELEASE TO PUBLIC	**19. SECURITY CLASS** *(This Report)* UNCLASSIFIED	**21. NO. OF PAGES** 258
	20. SECURITY CLASS *(This page)* UNCLASSIFIED	**22. PRICE** PCA12/mFA01

EPA Form 2220-1 (Rev. 4-77) PREVIOUS EDITION IS OBSOLETE

United States
Environmental Protection
Agency

Hazardous Waste Engineering
Research Laboratory
Washington DC 20460

EPA/600/2-87/065
August 1987

PB88-113295

Research and Development

EPA

Construction Quality Control and Post-Construction Performance Verification for the Gilson Road Hazardous Waste Site Cutoff Wall

REPRODUCED BY
U.S. DEPARTMENT OF COMMERCE
NATIONAL TECHNICAL
INFORMATION SERVICE
SPRINGFIELD, VA. 22161

EPA/600/2-87/065
August 1987

CONSTRUCTION QUALITY CONTROL AND POST-CONSTRUCTION PERFORMANCE VERIFICATION FOR THE GILSON ROAD HAZARDOUS WASTE SITE CUTOFF WALL

by

Matthew J. Barvenik & John E. Ayres
Goldberg-Zoino & Associates, Inc.
320 Needham Street
Newton Upper Falls, Massachusetts 02164

Project Officer

Stephen James
Land Pollution Control Division
Hazardous Waste Engineering Research Laboratory
Cincinnati, Ohio 45268

WATER SUPPLY AND POLLUTION CONTROL COMMISSION
STATE OF NEW HAMPSHIRE
CONCORD, NEW HAMPSHIRE 03301

HAZARDOUS WASTE ENGINEERING RESEARCH LABORATORY
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

NOTICE

The information in this document has been funded by the U.S. Environmental Protection Agency and the New Hampshire Water Supply and Pollution Control Commission under cooperative agreement Contract No. CR811130-01 with Goldberg-Zoino & Associates, Inc. It has been subjected to the Agency's peer review and administrative review, and it has been approved for publication as a U.S. EPA document. Mention of trade names or commercial products does not constitute an endorsement or recommendation for use.

FOREWARD

Today's rapidly developing and changing technologies and industrial products and practices frequently carry with them the increased generation of solid and hazardous wastes. These materials, when improperly dealt with, can threaten both public health and environment. Abandoned waste sites and accidental releases of toxic and hazardous substances to the environment also have important environmental and public health implications. The Hazardous Waste Engineering Research Laboratory helps provide an authoritative and defensible engineering basis for assessing and solving these problems. Its products support the policies, programs, and regulations of the Environmental Protection Agency; the granting of permits and other responsibilities of State and local governments; and the needs of both large and small businesses in handling their wastes responsibly and economically.

This report describe assessment activities undertaken to evaluate the effectiveness of a soil/bentonite backfilled cutoff wall (slurry trench) installed for the purpose of hazardous waste containment. The work includes development and rvaluation of field quality control tests, evaluation of electronic piezocone instrumentation for post-construction verification of backfill homogeneity, and evaluation of cutoff wall bulk hydraulic conductivity via hydraulic stress testing. This information in this report is useful to those involved in the feasibility study, design and/or construction of cutoff walls as a hazardous waste remediation technology.

For further information, please contact the Land Pollution Control Division of the Hazardous Waste Engineering Research Laboratory.

Thomas R. Hauser, Director
Hazardous Waste Engineering
Research Laboratory

ABSTRACT

This report is intended to provide a critical assessment of the construction and effectiveness of a soil/bentonite backfilled cutoff wall (slurry trench) installed for the purpose of hazardous waste containment. The wall was designed, constructed, and monitored as part of the remediation action implemented on the Gilson Road site in Nashua, New Hampshire (National Priority List [NPL] #23). This work was conducted in three distinct phases.

The first phase consisted of an evaluation of the accuracy, precision, and applicability of two new field quality control tests developed specifically to be used for control of wall construction. The API fixed ring test and the methylene blue titration test allowed real time determination of backfill hydraulic conductivity and percent bentonite, respectively. The resulting field data were assessed with respect to cutoff wall specification requirements and guidelines were developed.

The second phase of the work encompassed field testing of electronic piezocone instrumentation to determine feasibility for post-construction verification of backfill homogeneity. This entailed evaluation of piezocone response characteristics with respect to intact backfill versus non-backfill materials in a specially constructed full scale test section. Response signatures were then used to evaluate the homogeneity of the Gilson Road containment wall followed by verification via direct sampling.

The primary objective of the third phase of the project was to hydraulically stress the containment wall to assess its bulk hydraulic conductivity and thus its overall effectiveness with respect to hazardous waste containment. This work was supplemented with documentation of the spacial and temporal variation in contaminant concentrations as they reflect the behavior of the hydrologic system in response to containment wall installation.

This report is intended to document actual site specific data with respect to the effectiveness of a soil/bentonite cutoff wall used for hazardous waste containment. The data can then be used by those formulating and reviewing remedial action plans to rationally assess the suitability of cutoff walls incorporated as part of the overall site remediation proposed for other sites.

CONTENTS

Page

NOTICE.. ii
FOREWORD.. iii
ABSTRACT.. iv
CONTENTS.. v
FIGURES... ix
TABLES.. xii
ACKNOWLEDGEMENTS.. xiv

1. INTRODUCTION... 1-1
1.1 Overview.. 1-1
1.2 Background.. 1-3
  1.2.1 Site Description and Geology...................... 1-3
  1.2.2 Remedial Measures................................. 1-4
  1.2.3 Design and Construction........................... 1-5
1.3 Report Organization................................... 1-5

2. FIELD QUALITY CONTROL TESTING PROCEDURES................ 2-1
2.1 Evaluation of Backfill Hydraulic Conductivity Using the Fixed Ring Apparatus....................... 2-2
  2.1.1 API Fixed Ring Test............................... 2-4
  2.1.2 Testing Program................................... 2-6
    2.1.2.1 API Fixed-Ring Test Precision................. 2-6
    2.1.2.2 API Fixed-Ring Test Accuracy.................. 2-8
  2.1.3 Guidelines for Field use of the API Fixed-Ring Test.. 2-13
    2.1.3.1 Verification Testing.......................... 2-14
    2.1.3.2 Production Testing............................ 2-14
2.2 Evaluation of Backfill Bentonite Content Using Methylene Blue Titration............................... 2-17
  2.2.1 Methylene Blue Titration.......................... 2-18
  2.2.2 Testing Program................................... 2-20
    2.2.2.1 Distilled Water Matrix........................ 2-20
    2.2.2.2 Ottawa Sand/Distilled Water Matrix............ 2-23
    2.2.2.3 Contaminated Water Matrix..................... 2-25
    2.2.2.4 Gilson Road Backfill Matrix................... 2-27
  2.2.3 Field Use of the Methylene Blue Test.............. 2-31
2.3 Effect of Backfill Gradation on Hydraulic Conductivity.. 2-33
  2.3.1 Data Base... 2-34
  2.3.2 Statistical Evaluation............................ 2-35
  2.3.3 Importance of Statistical Correlation............. 2-37
2.4 Assessment of Gilson Road QC Data..................... 2-38
  2.4.1 Review of QC Data................................. 2-39
    2.4.1.1 QC Data from Construction Samples............. 2-39

CONTENTS (continued)

2.4.1.2 QC Data from Post-Construction Samples.............................. 2-43
2.4.1.3 Evaluation of the Validity of the QC Construction Data................. 2-47
2.4.1.4 Implication of QC Data on Containment Performance......................... 2-47
2.5 Quality Control Guidelines............................. 2-50
2.5.1 General Quality Control Guidelines............ 2-50
2.5.1.1 Test Requirements.................... 2-50
2.5.1.2 Quality Control Execution............ 2-51
2.5.2 QC Guidelines for Support Slurry.............. 2-53
2.5.2.1 Unit Weight.......................... 2-54
2.5.2.2 Viscosity............................ 2-55
2.5.2.3 Filtrate Loss........................ 2-55
2.5.3 QC Guidelines for Trench Excavation........... 2-56
2.5.3.1 Verticality.......................... 2-56
2.5.3.2 Continuity........................... 2-57
2.5.3.3 Key Penetration...................... 2-57
2.5.3.4 Key Cleaning......................... 2-58
2.5.3.5 Stability............................ 2-59
2.5.4 QC Guidelines for Backfill.................... 2-60
2.5.4.1 Unit Weight.......................... 2-60
2.5.4.2 Slump................................ 2-61
2.5.4.3 API Fixed-Ring Hydraulic Conductivity Testing.............................. 2-61
2.5.4.4 Methylene Blue Titration for Bentonite Content.................... 2-62
2.5.4.5 Gradation............................ 2-62
2.6 Conclusions.. 2-62

3. WINDOW DETECTION VIA PIEZOCONE SOUNDING.............. 3-1
3.1 Piezocone Operating Principles......................... 3-3
3.1.1 Standard Instrument Configuration............. 3-3
3.1.2 Instrument Modification....................... 3-5
3.1.3 Sounding Procedure............................ 3-7
3.1.4 Data Manipulation and Output Format........... 3-9
3.2 Material Identification................................ 3-10
3.2.1 Point Resistance.............................. 3-11
3.2.2 Local Friction................................ 3-12
3.2.3 Excess Pore Pressure.......................... 3-12
3.2.4 Relative Hydraulic Conductivity Index......... 3-15
3.2.5 Empirical Correlations........................ 3-17
3.2.6 Window Identification......................... 3-21
3.2.6.1 Intact Soil/Bentonite Backfill....... 3-22
3.2.6.2 Clean Granular Window................ 3-25
3.2.6.3 Slurry Filled Granular Window........ 3-27
3.2.6.4 Bentonite Support Slurry Filled Window............................... 3-28

CONTENTS (continued)

3.3 Cutoff Wall Test Section........................... 3-29
  3.3.1 Test Section Construction...................... 3-30
    3.3.1.1 Excavation................................. 3-30
    3.3.1.2 Backfilling................................ 3-33
    3.3.1.3 Window Emplacement......................... 3-34
  3.3.2 Test Section Window Verification............... 3-37
    3.3.2.1 High Density Clean Granular Window......... 3-38
    3.3.2.2 Low Density Clean Granular Window.......... 3-40
    3.3.2.3 Slurry Filled Granular Window.............. 3-40
    3.3.2.4 Support Slurry Filled Window............... 3-40
  3.3.3 Test Section Soundings......................... 3-42
    3.3.3.1 Intact Backfill Analysis................... 3-53
    3.3.3.2 High Density Clean Granular Window......... 3-58
    3.3.3.3 Low Density Clean Granular Window.......... 3-61
    3.3.3.4 Slurry Filled Granular Window.............. 3-65
    3.3.3.5 Support Slurry Filled Window............... 3-69
  3.3.4 Window Identification Guidelines............... 3-69
    3.3.4.1 High Density Clean Granular Window......... 3-70
    3.3.4.2 Low Density Clean Granular Window.......... 3-70
    3.3.4.3 Slurry Filled Granular Window.............. 3-71
    3.3.4.4 Support Slurry Filled Window............... 3-71
3.4 Containment Wall Soundings......................... 3-71
  3.4.1 Intact Backfill; Sounding GZ-13................ 3-73
  3.4.2 Potential Window Locations, GZ-13.............. 3-76
3.5 Window Verification................................ 3-81
  3.5.1 Sample Classification.......................... 3-82
    3.5.1.1 Boring GZA-13, 43 to 48 Feet............... 3-83
    3.5.1.2 Boring TBF-1A, 39 to 46 Feet............... 3-84
    3.5.1.3 Boring SU-7, 30.5 to 35.5 Feet, 45.5 to 48 Feet, and 49.5 to 52.0 Feet............... 3-84
    3.5.1.4 Boring SU-4, 21 to 26 Feet................. 3-85
  3.5.2 Sample Hydraulic Conductivity Testing.......... 3-86
  3.5.3 Containment Wall Homogeneity................... 3-86
3.6 Conclusions.. 3-87

4. EVALUATION OF CONTAINMENT LEAKAGE.................. 4-1
4.1 Bedrock Pumping Test............................... 4-2
  4.1.1 Pumping Well Location and Installation......... 4-3
  4.1.2 Recharge Trenches.............................. 4-5
  4.1.3 Monitoring Points.............................. 4-7
    4.1.3.1 Existing Points............................ 4-7
    4.1.3.2 New Installations.......................... 4-9
  4.1.4 Execution of Pump Test......................... 4-10
    4.1.4.1 General Conditions......................... 4-10
    4.1.4.2 Data Discussion............................ 4-13
  4.1.5 Analysis....................................... 4-18
  4.1.6 Conclusions.................................... 4-26

CONTENTS (continued)

4.2 Cutoff Wall Hydraulic Efficiency.................... 4-27
- 4.2.1 Test Constraints.............................. 4-28
- 4.2.2 Reference Data Set............................ 4-29
- 4.2.3 Numerical Methodology......................... 4-33
  - 4.2.3.1 Model Description.................... 4-34
  - 4.2.3.2 Model Calibration.................... 4-37
- 4.2.4 Sensitivity Analysis.......................... 4-39
- 4.2.5 Cutoff Wall Efficiency........................ 4-47
- 4.2.6 Summary....................................... 4-49

4.3 Groundwater Quality Monitoring...................... 4-50
- 4.3.1 Location and Installation of Groundwater Monitoring Wells............................ 4-50
- 4.3.2 Sampling and Analysis......................... 4-52
  - 4.3.2.1 Sampling of Monitoring Points........ 4-52
  - 4.3.2.2 Analysis of Groundwater Samples...... 4-52
- 4.3.3 Results of Groundwater Quality Monitoring..... 4-53
  - 4.3.3.1 Selection of Pertinent Groundwater Monitoring Points.................... 4-53
  - 4.3.3.2 Selection of an Indicator VOC Compound............................. 4-55
  - 4.3.3.3 Data Discussion...................... 4-57
  - 4.3.3.4 Contaminant Degradation of the Cutoff Wall.......................... 4-61

4.4 Conclusions... 4-64

5. CONCLUSIONS... 5-1

REFERENCES.. 6-1

APPENDICES 7-1

- A. API FIXED-RING PROCEDURE........................ 7-1
- B. METHYLENE BLUE TEST PROCEDURE................... 8-1
- C. GEOPHYSICAL METHODS FOR WINDOW DETECTION......... 9-1

LIST OF FIGURES

Number		Page
2-1	API Apparatus for Hydraulic Conductivity Testing	2-5
2-2	Hydraulic Conductivity: API vs. Triaxial (Remixed)	2-11
2-3	Triaxial Hydraulic Conductivity Results: "Undisturbed" vs. "Remixed" Samples	2-16
2-4	Methylene Blue Test Results: Bentonite in Distilled Water	2-22
2-5	Methylene Blue Test Results: Bentonite in Ottawa Sand/Distilled Water	2-24
2-6	Methylene Blue Test Results: Bentonite in Contaminated Water	2-26
2-7	Methylene Blue Test Results: Bentonite in Borrow/Trench Spoil/Distilled Water	2-30
2-8	Hydraulic Conductivity vs. Time - Bentonite Content vs. Time	2-40
2-9	Hydraulic Conductivity vs. Station - Bentonite Content vs. Station	2-42
2-10	Apparent Zones of Backfill with Hydraulic Conductivity Greater than 1 x 10^{-7} cm/sec	2-49
3-1	Schematic of Piezocone Instrument	3-4
3-2	Schematic of Permeameter	3-6
3-3	Permeameter Pressurization/Flow Measurement System	3-8
3-4	Excess Pore Pressure Dissipation (Wissa, 1975)	3-16
3-5	Material Identification-Friction Ratio	3-18
3-6	Material Identification-Friction Ratio	3-19
3-7	Material Identification-Excess Pore Pressure	3-20

LIST OF FIGURES (continued)

3-8 Relative Piezocone Response.................... 3-23

3-9 Location Plan.................................. 3-32

3-10 Test Section Construction Profile.............. 3-35

3-11 Post-Construction Test Section Profile.......... 3-39

3-12 Piezocone Test--Sounding No. GZ1................ 3-43

3-13 Piezocone Test--Sounding No. GZ2................ 3-44

3-14 Piezocone Test--Sounding No. GZ3................ 3-45

3-15 Piezocone Test--Sounding No. GZ4................ 3-46

3-16 Piezocone Test--Sounding No. GZ5................ 3-47

3-17 Piezocone Test--Sounding No. GZ6................ 3-48

3-18 Piezocone Test--Sounding No. GZ7................ 3-49

3-19 Piezocone Test--Sounding No. GZ8................ 3-50

3-20 Piezocone Test--Sounding No. GZ9................ 3-51

3-21 Piezocone Test--Sounding No. GZ10............... 3-52

3-22a Piezocone Test--Sounding No. GZ13............... 3-74

3-22b Piezocone Test--Sounding No. GZ13............... 3-75

4-1 Pumping Test Monitoring Network................. 4-4

4-2 Pumping Well Installation....................... 4-6

4-3 Typical Existing Multilevel Installation........ 4-8

4-4 Typical Pumping Test Multilevel Installation..... 4-11

4-5 PT-2 Time-Drawdown Curve........................ 4-14

4-6 M-14 Time-Drawdown Curve........................ 4-15

4-7 M-3-4/M-3-5 Time-Drawdown Curve................. 4-17

4-8 Measured Bedrock Drawdown Contours.............. 4-19

LIST OF FIGURES (continued)

4-9 Measured Overburden Drawdown Contours............ 4-20

4-10 Pumping Test Idealized Geometry.................. 4-21

4-11 Modeled Bedrock Drawdown Contours................ 4-24

4-12 Modeled Overburden Drawdown Contours............. 4-25

4-13 Measured Water Table Contours, June, 1983........ 4-30

4-14 Vertical Head Distribution at M-10R 1 M-14, June, 1983.. 4-32

4-15 Cutaway View of Numerical Model.................. 4-35

4-16 Plan View of Top Layer of Model.................. 4-36

4-17 Jan. 82 Scenario Calibration..................... 4-40

4-18 June 83 Scenario Calibration..................... 4-41

4-19 Oct. 83 Scenario Calibration..................... 4-42

4-20 Sept. 85 Scenario Calibration.................... 4-43

4-21 Modeled Vertical Head Distributions for Different Wall Conductivities.................. 4-45

4-22 Location of Groundwater Monitoring Points........ 4-51

4-23 Location of Selected Groundwater Monitoring Points... 4-56

4-24a Variations in Contaminant Distribution........... 4-58

4-24b Variations in Contaminant Distribution........... 4-59

LIST OF TABLES

Number		Page
2-1	Results of Hydraulic Conductivity Tests in API Fixed-Ring Apparatus: Precision Series	2-7
2-2	Hydraulic Conductivity Testing: API Fixed-Ring Vs. Triaxial (Remixed)	2-10
2-3	Effect of Varying Gradient in API Fixed-Ring Test	2-13
2-4	Triaxial Hydraulic Conductivity: "Undisturbed" vs. "Remixed" Samples	2-15
2-5	Methylene Blue Test Results On-Site (Trench Spoil)	2-28
2-6	Methylene Blue Test Results Off-Site Borrow	2-29
2-7	Correlation Coefficients - Hydraulic Conductivity vs. Gradation Parameters	2-36
2-8	Hydraulic Conductivity: "Undisturbed" vs. "Remixed" Samples	2-45
3-1	Support Slurry QC Data	3-31
3-2	Soil/Bentonite Backfill QC Data	3-33
3-3	Intact Backfill	3-54
3-4	Relative Hydraulic Conductivity Index	3-55
3-5	High Density Clean Granular Window	3-59
3-6	Low Density Clean Granular Window	3-62
3-7	Slurry Filled Granular Window	3-66
4-1	Daily Precipitation During Pumping Test (9/85)	4-13
4-2	Stress Scenarios and Simulation Fluxes	4-38
4-3	Calibration Deviation Statistics	4-39

LIST OF TABLES (continued)

4-4 Modeled Outward Fluxes.............................. 4-48

4-5 Sensitivity Run Fluxes.............................. 4-49

4-6 Concentrations of Tetrahydrofuran in Parts Per Million.................................. 4-57

ACKNOWLEDGEMENTS

This document was prepared by Goldberg-Zoino & Associates, Inc. in partial fulfillment of contract No. CR811130-01, which was funded under a cooperative agreement between the New Hampshire Water Supply and Pollutant Control Commission and the U.S. EPA's Hazardous Waste Engineering Research Laboratory. The state of New Hampshire's project officer was Michael Donahue with major input from Michael Sills. The EPA project officer was Stephen James. Initial input was provided by Dr. Walter Grube and Jon Herrmann, also of the U.S. EPA's Hazardous Waste Engineering Research Laboratory.

The Principal-in-Charge of the project for Goldberg-Zoino & Associates, Inc. was John Ayres; Matthew Barvenik was Project Manager and principal author. Other GZA contributing authors included William Hadge, David Brown, Thomas Kern, and Brian Dorwart.

The following individuals provided direct input to the project via laboratory work, field observation, data analysis, etc., during the design and construction of the Gilson Road cutoff wall and/or during execution of the project:

Michael Powers	GZA
Charles Lindberg	GZA
Donald Schulze	GZA
Katrina Grundstrom	GZA
John Forde	GZA
Tim Cady	GZA
Paul Hertzlier	State of New Hampshire
Ralph Wickson	State of New Hampshire
Bob Ostrofsky	State of New Hampshire
Dan Tiltgis	Case International (now with Recosol, Inc.)
Joe Lewis	Case International
James Handly	James Handly Company

SECTION 1

INTRODUCTION

The work performed under this contract was funded under a cooperative agreement between the New Hampshire Water Supply and Pollution Control Commission and the U.S. EPA Hazardous Waste Engineering Research Laboratory. As a cooperatively funded project, two separate but complementary objectives were established. The first, and primary, objective was to evaluate the design, construction, operation, and monitoring of the Gilson Road National Priority List site remediation. As such, this information enhanced the overall effectiveness of the passive and active barrier containment system. The second objective was to conduct an evaluation which would lead to a better understanding of containment technologies in general. This knowledge could then be incorporated into the design and construction of hazardous waste remediation actions on other NPL sites.

1.1 Overview

Soil/bentonite backfilled cutoff walls have successfully been used as an adjunct to pumping for excavation dewatering and general groundwater control for over forty years. The procedure of adding bentonite to excavated in-situ soil and then placing the low hydraulic conductivity mixture back in the slurry-supported trench to form a barrier was initially used in Europe and then in the United States. Specifications governing this work typically fell under the dewatering section of contract construction documents and, as such, were performance oriented. In many cases, success was judged solely on the ability of the barrier to limit fluid flux to quantities that could reasonably be pumped. Due to the magnitude of the overall construction effort typically involved, the addition of pumping to remove leakage from the excavation with off-site disposal was often considered insignificant.

More recently, soil/bentonite (S/B) cutoff technology has been used as part of remedial actions at United States Environmental Protection Agency National Priority List (NPL) sites, as well as private industrial sites. (JRB, 1984). For these more critical applications, leakage must be reduced to a far greater extent than typically associated with construction dewatering.

The successful construction of a hazardous waste cutoff wall containment is largely dependent on the quality of the contractor's workmanship. As such, specifications must remain generally performance oriented with the primary requirement of attaining a long-term hydraulic conductivity less than some specified value; typically less than 1 x 10^{-7} cm/sec for waste containment. However, in recognition of the difficulties involved in performance verification and the consequences of detecting substandard construction only after wall completion, contract documents must also include minimum design standards. These center around a number of design and construction issues such as minimum bentonite and fines content in the backfill, for example (Ayres, 1983). This allows quality control (QC) testing to be executed in the field during construction as a check on attainment of the minimum standards specified. "Learning Curve" problems can therefore be detected and addressed early in the construction process before they become pervasive and jeopardize the overall effectiveness of the barrier. Adequate specification and quality control of such minimum design standards requires intimate knowledge of the technology underlying cutoff wall construction as tempered with practical empiricism gained via construction experience.

As is often the case with initial technology transfer, cutoff walls have been designed and specified by professionals associated with the hazardous waste field, rather than those experienced in cutoff wall design/construction. Environmental professionals, although familiar with the hydrogeologic constraints of the application, generally lack expertise and experience in cutoff wall specific technology. Specifications have therefore been strictly performance based in some cases, thus allowing contractors to execute the work in a manner consistent with past dewatering practices. Unfortunately, fluid flux rates associated with standard construction practices for dewatering purposes are unacceptable with respect to hazardous waste containment. As a result, some of these containments have not performed up to expectation. Cutoff walls have thus fallen from favor with respect to hazardous waste remediation due to poor implementation rather than limitations inherent in the technology.

Use of procedures, such as field QC, to prevent cutoff wall deficiencies before they occur is not only highly desirable but necessary for hazardous waste containment applications. However, it must also be recognized that the specifications under which these walls are constructed are principally performance based. The performance clauses incorporated in the specifications can only be meaningful to the extent that equipment and methods are available to enforce them, should post-construction verification become necessary. A demonstrated capability for performance verification also instills added incentive to achieve the highest quality of workmanship possible during construction. As such, instrumentation and procedures should be developed to identify unsatisfactory zones in completed cutoff walls as well as hydraulically test their post-construction performance.

1.2 Background

The Gilson Road uncontrolled hazardous waste site in Nashua, New Hampshire was the subject of the first site remediation cooperatively funded under EPA's Superfund program. Clandestine dumping of toxic volatile organic compounds into an abandoned gravel pit resulted in a contaminant plume which degraded residential air quality as the contaminants volatilized following recharge to a local stream and threatened downstream municipal drinking water supplies (Barvenik, 1986).

1.2.1 Site Description and Geology

The site is underlain by glacial soils consisting of primarily granular ice-contact and glacial lake deposits. These include fine to coarse sands and gravels, with layers and lenses of silts and silty fine sands. Below the stratified drift is a granular basal till. This, in turn, is underlain by a moderately fractured schist bedrock, with a weathered, highly fractured upper zone (Koteff, 1973).

The sand and sand/gravel deposits exhibit an average hydraulic conductivity of 10^{-2} cm/sec based on pumping test results. However, these deposits vary over a range from 10^{-5} cm/sec for the silts and silty fine sands to an estimated 10^{-1} cm/sec for open work gravel seams found in the esker deposits. The glacial till deposits are typically less permeable (10^{-5} cm/sec) than the average values for the overlying stratified drift. However, the till is generally thin and discontinuous in nature, thus constituting a relatively imperfect aquitard. The hydraulic conductivity of the upper 50 feet of the bedrock was estimated to range between 10^{-5} and 10^{-3} cm/sec, based on investigation phase packer testing. The bedrock pumping test executed as part of this post-construction project, however, indicated that the upper 10 to 15 feet is shattered, with a hydraulic conductivity of over 10^{-1} cm/sec. This condition exists at least locally, in the vicinity of the downgradient wall.

1.2.2 Remedial Measures

Final designed remedation for the site consisted of complementary active and passive physical barriers to control groundwater flow, and treatment of the captured contaminant plume. This interactive system was adopted because neither a purely passive barrier nor a purely active barrier alone could eliminate off-site contaminant migration. A cutoff wall to the top of bedrock and a synthetic cap form the passive elements of the system. The cutoff wall was 4,000 feet in length, up to 110 feet deep, and completely encompassed the entire 20 acre site. Extending the passive barrier through the fractured portion of the bedrock was evaluated during initial design phases. This could be accomplished by excavating through the bedrock or grouting the rock fractures. Albeit feasible, neither method was found to be a cost-efficient solution for limiting contaminant flux through the bottom of the containment.

The active component of the total containment vessel is a hydrodynamic isolation system which accounts for the inadequacies of the bedrock as an aquiclude. This system redistributes the groundwater within the containment area and eliminates head differences across the cutoff wall to achieve a no leakage condition (zero gradient). The groundwater extraction/recharge associated with the hydrodynamic isolation also allows for treatment of the recirculated water, resulting in aquifer restoration. The restoration process requires a number of flushing cycles for complete contaminant removal.

A purely active system of plume control via hydrodynamic extraction/recharge was also found to be unacceptable during the feasibility study due to the heterogeneity of the aquifer. Groundwater extraction at a rate necessary to eliminate plume migration would necessarily entail the pumping of significant volumes of clean water from regions of high hydraulic conductivity outside the plume and nearby surface waters. This dilution of the plume at the point of recharge and the concomitant spreading quickly results in a contaminant plume width greater than the extraction system's radius of influence. Even in a homogeneous aquifer, contaminants can still escape active containment if the recharge point does not lie directly upstream of the extraction well (i.e., along a flow line). Finally, even with a perfect recirculation system, contaminant dispersion across the flow lines at the edge of the containment results in a further loss of contaminant mass from the system.

Although the cutoff wall provides enhanced hydraulic control, no practical hydrodynamic isolation system incorporating a finite number of extraction and recharge points, can completely eliminate all gradients across the physical barrier. Therefore, to preclude off-site contaminant migration, the system is operated with a conservative bias by forcing an inward gradient everywhere along the cutoff wall. The required net recharge deficit, relative to extraction, necessitates an external purge stream (discharged outside of the containment). The purge stream volume is, in part, a function of the hydraulic conductivity of the containment perimeter (cutoff wall). To attain an extremely high water quality in the purge stream, as mandated for off-site disposal at this CERCLA site, this portion of the pumpage must undergo additional treatment steps as compared to the total recirculation flow. Therefore, the hydraulic efficiency of the cutoff wall directly affects purge stream treatment costs, which are a considerable fraction of the two million dollar/year overall operation and maintenance cost of the on-site treatment facility.

1.2.3 Design and Construction

In recognition of the importance of the cutoff wall to the successful operation of the hydrodynamic isolation/recirculation system, a rigorous design phase was implemented including an advanced soil/bentonite backfill design methodology (Schulze, 1984) and development of new equipment and procedures for quality control field testing. This work was incorporated into the specifications as minimum design standards keyed to specific aspects of the actual construction process (Ayres, 1983). The Gilson Road project therefore represents perhaps the first case where the level of cutoff wall design, specification, and construction quality control (Q/C) were commensurate with application to hazardous waste containment. As such, this site was considered ideal for use in evaluation of the suitability of soil/bentonite cutoff wall technology as part of an overall containment strategy for hazardous waste remediation.

1.3 Report Organization

The scope of this work was separated into three major phases. The first phase (Section 2) encompassed evaluation of two new testing techniques used for quality control during construction of the Gilson Road containment. The methylene blue titration and the API fixed ring hydraulic conductivity tests were employed to determine the bentonite content and hydraulic conductivity of the intact soil/bentonite cutoff wall backfill, respectively. The project described herein demonstrated the applicability, accuracy, precision, and practicality of these tests for quality control.

The second phase of this project (Section 3) was directed at investigation of the feasibility of piezocone instrumentation for verification of cutoff wall backfill homogeneity. This entailed evaluation of piezocone response characteristics with respect to intact backfill versus non-backfill materials in a specially constructed full scale test section. Response signatures were then used to evaluate the homogeneity of the Gilson road containment wall followed by verification via direct sampling.

The primary objective of third phase of the project (Section 4) was to hydraulically stress the containment wall to assess its bulk hydraulic conductivity and thus its overall effectiveness with respect to hazardous waste containment. This work was supplemented with documentation of the spacial and temporal variation in contaminant concentrations as they reflect the behavior of the hydrologic system in response to containment wall installation.

Each of the three above described major sections are organized to be independent, self-contained documents. The conclusions derived from the work are therefore presented after each individual section. The report concludes with a final section summarizing the general conclusions which can be drawn from this project.

SECTION 2

FIELD QUALITY CONTROL TESTING PROCEDURES

Quality control testing of soil/bentonite backfill during cutoff wall construction is essential from two perspectives: 1) it allows an on-going evaluation of construction procedures; and 2) test results form the basis of evaluating future wall performance. QC testing provides a means of assessing whether minimum design standards have been met for a discrete number of representative backfill samples obtained prior to placement in the trench. From this set of data, future performance of the wall as a whole is extrapolated. While recognizing that wall performance may be significantly impaired by anomalies in the backfill not revealed by the QC data, QC testing imposes control over a construction procedure for which there are few visual indicators of quality. Emphasis of QC testing during construction thus allows for the timely and efficient correction of deficiencies. Over the long-term, QC testing during construction also reduces the need for post-construction verification of backfill parameters--which is costly--and the reliance on post-construction "leakage data"--which can be circumstantial and subject to varied interpretation. Moreover, even if deficiencies can be detected in the completed wall, their correction is generally neither simple nor inexpensive.

An effective QC program involves frequent testing of primary backfill parameters, including unit weight, slump, gradation, fines content, bentonite content, and hydraulic conductivity. The real time data obtained through field QC testing are made available to the contractor to allow modification of component proportioning and mixing effort at the outset of the project to achieve specified requirements. Standard methods for determining unit weight, slump, fines content, and overall gradation are easily executed in the field within a time period commensurate with real time QC of field operations (typically within one day). However, long periods of time are required for triaxial testing for hydraulic conductivity (greater than one week) and standard hydrometer testing of bentonite content (typically three to four days). In recognition of these limitations, new S/B backfill QC techniques, including API fixed-ring permeation, and methylene blue titration, were utilized on the Gilson Road project for bentonite content and hydraulic conductivity determinations, respectively. This work demonstrated the applicability and practicality of these new methods for field control.

The first phase of the project, presented in this chapter, focused on the refinement of these two new QC procedures and evaluation of their precision and accuracy using samples obtained from the Gilson Road site (Sections 2.1 and 2.2). Guidelines for field use were developed. To better assess the role of gradation parameters on hydraulic conductivity, a statistical evaluation of the QC data obtained from the Gilson Road project was also performed (Section 2.3). The QC data obtained from the Gilson Road project were then reviewed to assess the conformance of the completed wall to project specifications (Section 2.4). Finally, quality control guidelines for cutoff wall construction were developed, encompassing trench excavation and backfill placement as well as backfill preparation (Section 2.5).

2.1 Evaluation of Backfill Hydraulic Conductivity Using the Fixed-Ring Apparatus

The primary hydrologic feature of a cutoff wall is its low hydraulic conductivity and thus high impedance with respect to groundwater flow. This characteristic results in the effectiveness of cutoff walls as physical barriers to contaminant migration. In recognition of the importance of the hydraulic conductivity actually attained during cutoff construction, this key parameter is subject to a high degree of specification in project contract documents. For most hazardous waste containments, an upper bound hydraulic conductivity of 1 x 10-7 cm/sec is typically required, as was the case for the Gilson Road site.

A number of criteria must be met during construction to achieve a soil/bentonite cutoff wall with the specified hydraulic conductivity. The types and percentages of backfill components have a large effect on hydraulic conductivity and are typically determined via laboratory testing undertaken during the design phase (Schulze, 1984). These data are generally reflected in the project specifications as minimum design standards. (For example, the Gilson Road specifications required a minimum of 30 percent fines and 5 percent minimum bentonite content for the backfill). Even if these minimum criteria are met, however, the proper hydraulic conductivity is not guaranteed due to the importance of uniform mixing of the backfill components, particularly with respect to dispersion of the bentonite. Inasmuch as proper proportioning of backfill components and sufficient mixing effort are primarily related to the quality of workmanship on the part of the contractor, the required hydraulic conductivity for the cutoff appears as a performance specification in the project documents.

To have any impact as a specification requirement, performance criteria should be subject to immediate verification. However, in the strictest sense, the actual long-term hydraulic conductivity of the cutoff wall can only be evaluated through post-construction monitoring. This is a costly endeavor due to the requirement for closely-spaced multilevel piezometric head and groundwater quality monitoring installations located around the containment perimeter, both on the outside and inside of the wall. In all but the simplest cases, the data thus obtained also require interpretational techniques based on three-dimensional computer simulation.

Beyond the high cost, the major drawback associated with post-construction monitoring is that zones of unacceptable hydraulic conductivity are detected only after it is too late for prevention and/or cost-efficient repair. Therefore, while performance criteria are still necessary for specification of hydraulic conductivity, a more cost-effective and timely procedure is required for prediction, and thus construction control, of the hydraulic conductivity finally achieved by the barrier. This function can be fulfilled via QC testing as demonstrated on the Gilson Road project.

Though not an absolute determinant of the long term, in-place performance of the overall cutoff, hydraulic conductivity testing of the backfill prior to placement in the trench provides a good indicator of the eventual intact hydraulic conductivity of the barrier. Standard triaxial testing has therefore been routinely employed to provide these data despite its limitations. Due to the sophistication of the equipment required, the tests are costly (typically greater than several hundred dollars/test) and cannot easily be performed in the field. As such, a statistically insignificant number of tests are generally undertaken (normally not more than one test per multiple thousands of cubic yards of backfill) and the data become available only after several weeks have elapsed since the samples were obtained. This procedure suffers from the syndrome of "too little and too late" with respect to influence on the eventual performance of the cutoff barrier. Hence, it can only be viewed as a token post-construction verification effort.

An alternate procedure for obtaining hydraulic conductivity data employs fixed-ring equipment. Although, in general, not as accurate as triaxial testing, fixed-ring tests yield acceptable data if the correct procedures are followed to limit boundary leakage and the samples are conformal and initially saturated, or nearly so (Schulze, 1984).

Fixed-ring equipment was used to provide hydraulic conductivity data during construction of the Gilson Road cutoff wall. This inexpensive, easily-operated equipment allowed field testing of numerous samples prior to permanent incorporation of the backfill into the cutoff. The one day availability of hydraulic conductivity data allowed real-time quality control of the construction process. In light of the high degree of usefulness and practicality of the API fixed-ring hydraulic conductivity testing as shown on the Gilson Road project, a testing program was undertaken to:

- Assess the precision of the test
- Assess the accuracy of of the test.
- Refine and standardize the test procedure for incorporation in general guidelines for QC testing of S/B cutoff walls.

The test program and test results are described in the following subsections.

2.1.1 API Fixed-Ring Test

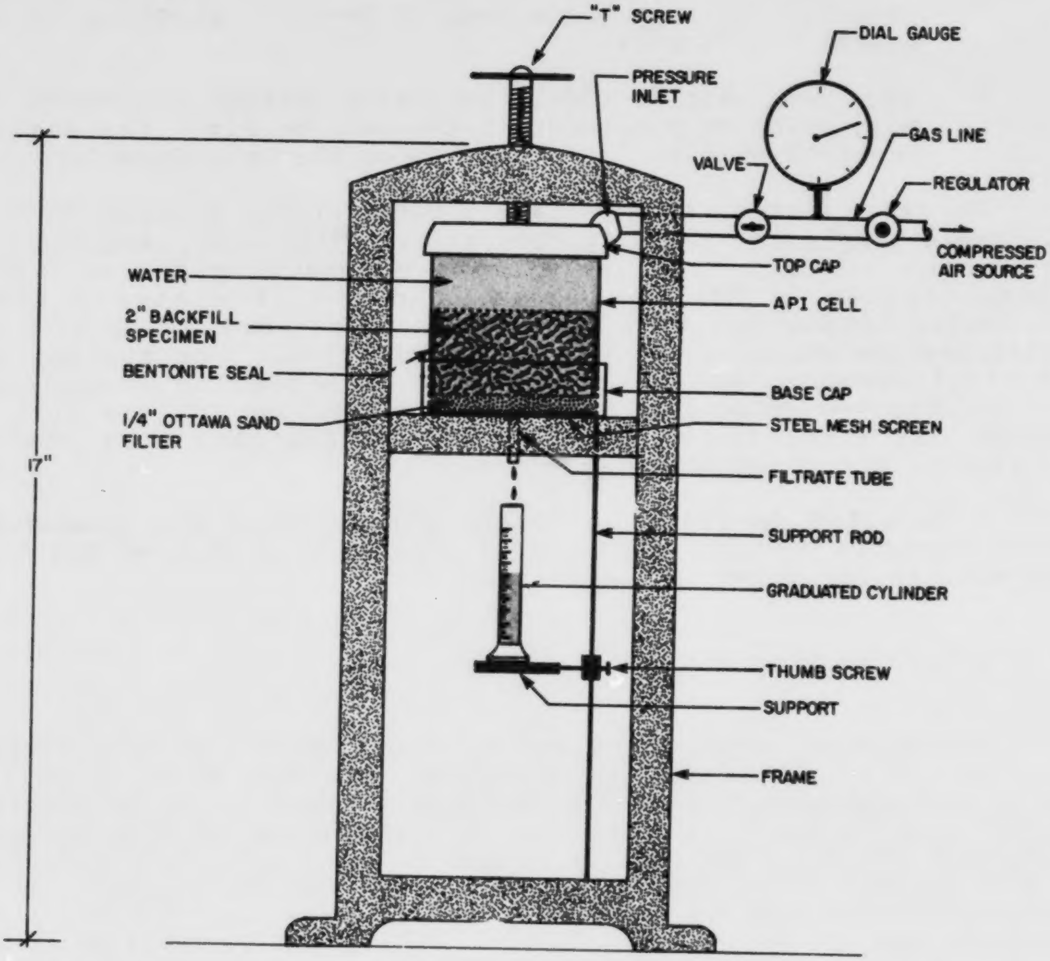
Various equipment configurations based on fixed-ring confinement of a test sample for evaluation of hydraulic conductivity have been in use for many years (U.S. Army Corps manual, 1970 and Lambe, 1951). In most cases, however, the equipment is meant for laboratory use and not designed to withstand the "environment" associated with field testing.

In an effort to standardize procedures around equipment which would survive repeated field use and also be readily obtainable, the conventional American Petroleum Institute (API) filter press was adopted for use as a fixed-ring permeameter (see Figure 2-1). As a filter press, this equipment is already routinely used on cutoff wall projects to test the filter cake forming capacity of bentonite slurries used for trench stabilization.

To allow accurate testing of S/B backfill samples at stresses commensurate with in-situ conditions, a number of minor equipment and procedural modifications are required, including:

- The pressure system should be modified to accept a regulator for each cell along with a gage which has a range of 1 to 15 psi for application of low pressure.

FIG. 2-1 A P I. APPARATUS FOR HYDRAULIC CONDUCTIVITY TESTING


"T" SCREW
DIAL GAUGE
PRESSURE INLET
GAS LINE
VALVE
REGULATOR
COMPRESSED AIR SOURCE
TOP CAP
WATER
API CELL
2" BACKFILL SPECIMEN
BENTONITE SEAL
BASE CAP
1/4" OTTAWA SAND FILTER
STEEL MESH SCREEN
17"
FILTRATE TUBE
SUPPORT ROD
GRADUATED CYLINDER
THUMB SCREW
SUPPORT
FRAME

(APPARATUS AS SHOWN MANUFACTURED BY BAROID)

- Bentonite "paste" should be applied to the inside diameter of the rigid wall and trimmed to a uniform thickness of 1/32 inch using a trimming jig. This will limit boundary flux along the backfill-rigid wall interface.
- Test samples should be fabricated on a thin bed of sand underlain by a porous screen to prevent plugging of the bottom outlet port.
- The test samples should be sieved through a standard 1/2 mesh prior to placement in the cell to limit the maximum particle size to less than 1/6 of the cell diameter.

In basic terms, API hydraulic conductivity testing involves placing a sample in the cell, covering it with water, and forcing the water through the sample using pneumatic pressure. Once consolidation of the sample is complete, the steady state hydraulic conductivity is computed based on standard Darcy's Law relationships assuming 100 percent saturation. For typical S/B backfill samples, approximate values of hydraulic conductivity can be obtained in several hours and final values in less than 24 hours. As such, the procedure is ideal for real time quality control of the construction process.

A detailed description of the standardized test procedure, which directly reflects the results of efforts performed for this project, is presented in Appendix A.

2.1.2 Testing Program

The overall program objective was to determine the inherent reliability of the API fixed-ring test for measuring the hydraulic conductivity of S/B backfill samples. This information could then be used to evaluate the validity of the QC data obtained during construction of the Gilson Road cutoff and, from a more general standpoint, form a basis for standardization of QC procedures for use on other projects. More specifically, the project was structured to first evaluate the precision of the test and then assess test accuracy. These two phases of the work are described in the following subsections.

2.1.2.1 API Fixed-Ring Test Precision

The precision testing was directed towards determining if equipment and procedures as simple as the API-fixed ring-based QC testing performed on the Gilson Road project would provide repeatable hydraulic conductivity values.

Five batches of the Gilson Road backfill were selected from record samples obtained during construction. Selection emphasized encompassing the full range of gradations and hydraulic conductivities encountered during construction of the actual cutoff wall. After thorough remixing in the laboratory, three "identical" aliquots of each sample were tested at a gradient of 40 (driving pressure of 3 psi - all pressures reported as gage). The results are presented in Table 2-1.

TABLE 2-1

RESULTS OF HYDRAULIC CONDUCTIVITY TESTS IN API FIXED-RING APPARATUS: PRECISION SERIES

Sample	Hydraulic Conductivity cm/sec (3 splits of each sample)
Station 0+50	0.48×10^{-7}
	0.38×10^{-7}
	0.46×10^{-7}
Station 7+75	4.02×10^{-7}
	4.02×10^{-7}
	4.44×10^{-7}
Station 14+75	0.80×10^{-7}
	1.86×10^{-7}
	0.79×10^{-7}
Station 20+25	0.63×10^{-7}
	0.25×10^{-7}
	0.25×10^{-7}
Station 35+00	0.69×10^{-7}
	0.28×10^{-7}
	0.33×10^{-7}

The data indicate a worst case precision of ±57 percent within one standard deviation of the mean value for any group of three tests. Although this initially appears as a large degree of scatter, it is actually quite small when compared to the five orders of magnitude (100,000 times) reduction in hydraulic conductivity of the initial in-situ soils. Further inspection of the data also reveals that in all five groups of tests, at least two of the three tests in each group agreed to ±10 percent within one standard deviation of the mean. Hence, some of the scatter reflected in the worst case precision of ±57 percent may be attributable to variations in aliquots of the same sample that were not eliminated by mixing. It must be recognized that the degree to which the precision of a test procedure can be established is highly dependent on how identical the test specimens are prepared. This is problematic for API fixed-ring hydraulic conductivity testing in that a single specimen cannot be set up more than once. Therefore, multiple specimens were required to evaluate the degree of variability in the entire procedure, from set-up through permeation. Unfortunately, the coarseness of granular based backfills such as used on the Gilson Road project can result in a somewhat heterogeneous sample from which the individual aliquots are taken for the comparison testing. Therefore, uniform mixing, to the extent practical in the laboratory, was emphasized during sample preparation. However, it is likely that some degree of heterogeneity still existed among the comparison samples.

This hypothesis is supported by field data obtained during subsequent QC testing for other cutoff walls utilizing primarily clayey mixes. For these finer grained backfills, the probability of obtaining identical samples is significantly enhanced. API fixed-ring tests run on "identical" samples of clayey mixes yielded closer precision than for the Gilson Road backfill.

2.1.2.2 API Fixed-Ring Test Accuracy

The accuracy of the API fixed-ring test was evaluated via comparison with triaxial tests. Inasmuch as the value of hydraulic conductivity attributed to a backfill specimen is dependent on the method of measurement, determination of the accuracy of a specific testing procedure suffers from the lack of an absolute value of hydraulic conductivity for comparison. Even triaxial testing, which is one of the most well developed and flexible testing techniques for determining hydraulic conductivity, can yield a wide range of values for the same S/B backfill sample. This variability stems primarily from the lack of standardization of physical testing parameters such as state of stress, degree of saturation, gradient, etc., to which the sample is highly sensitive (Schulze, 1984).

The accuracy evaluations made herein are therefore based on "absolute" values determined via triaxial procedures in which the physical testing parameters were standardized to model in-situ conditions at the top of the cutoff wall (upper 5 to 10 feet). As such, these "absolute" values of hydraulic conductivity would be conservative (high) relative to those expected for the same sample tested under conditions simulating greater depths or incomplete saturation. The philosophy of modeling the worst case condition of "top of wall" recognizes the indeterminancy of the final vertical location of a specific batch of backfill represented by a QC sample taken prior to backfill placement.

During construction of the Gilson Road cutoff wall, ten samples of the backfill were obtained specifically for triaxial testing. Each sample was remixed to improve uniformity and consistency and then split to permit field testing in the API fixed-ring apparatus as part of the QC program. In addition, five samples of backfill were obtained from the completed wall via conventional stationary-piston, tube sampling techniques for both API fixed-ring and triaxial hydraulic conductivity testing. Each of these samples was remixed prior to triaxial testing and then a split of the triaxial sample was used for API fixed-ring testing. Therefore, to the degree that the fifteen samples taken were homogeneous, the value of hydraulic conductivity was determined by both triaxial and API fixed-ring procedures on "identical" specimens. These data are presented on Figure 2-2 and are summarized in Table 2-2.

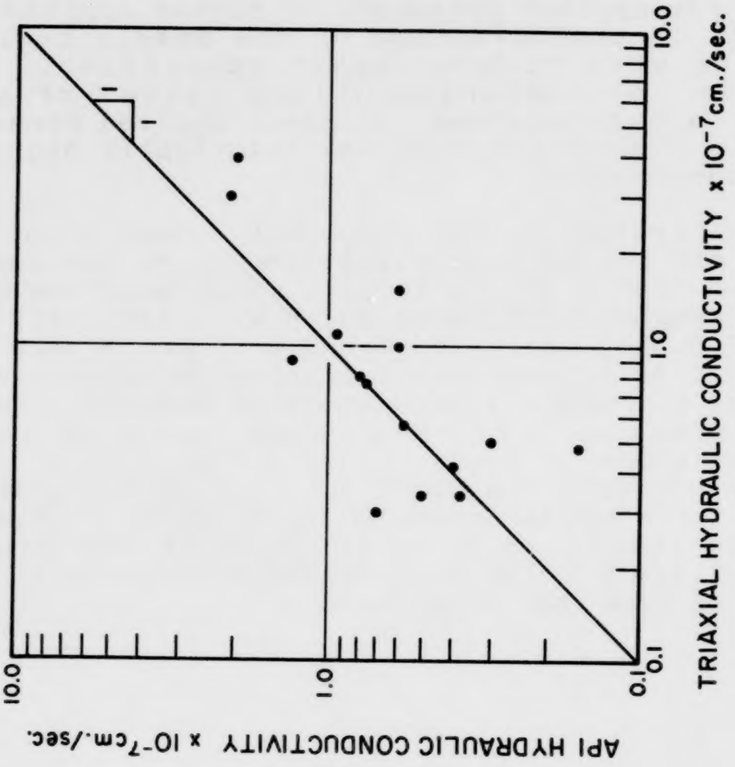
TABLE 2-2

HYDRAULIC CONDUCTIVITY TESTING: API FIXED-RING VS TRIAXIAL (REMIXED)

Station	API x 10^{-7} cm/sec	Triaxial x 10^{-7} cm/sec
0+00	0.93	1.1
1+00	0.50	1.0
1+00	0.80	0.80
3+45	1.0	4.0
3+45	0.80	1.5
4+75	0.38	0.34
9+00	2.1	3.0
11+00	0.70	0.30
12+50	0.50	0.34
16+50	0.40	0.42
20+50	0.57	0.56
24+50	0.76	0.75
28+50	1.3	0.90
32+50	0.16	0.48
36+50	0.30	0.50

The data indicate that the API fixed-ring values match the triaxial values to within ±66 percent in fourteen out of the fifteen cases, with one case of the API fixed ring value 133 percent higher than the triaxial. This error band not only encompasses the accuracy of the API fixed-ring test but also incorporates errors associated with heterogeneity between "identical" samples and precisional errors inherent in both the

FIG. 2-2 HYDRAULIC CONDUCTIVITY: API. vs. TRIAXIAL (REMIXED)



API fixed-ring and triaxial procedures. The error associated with heterogenity between "identical" samples could not be quantified. Precisional error for API fixed-ring testing was found to be ±57 percent. Precisional error for triaxial testing was not quantified, but experience suggests an error of ±50 percent. As such, the data indicate a very good level of agreement between the simple field API fixed-ring test and the more sophisticated laboratory triaxial test.

It should be noted that the applied pressure in thirteen of the fifteen API fixed-ring tests described above was 10 psi. The other two tests were run at 2 psi. Ten psi is considered an upper limit for applied pressure. A higher applied pressure would result in consolidation of the sample to levels not commensurate with "top of wall" conditions. As such, artificially low (unconservative) values of hydraulic conductivity could be obtained. A higher applied pressure could also lead to piping and thus unrealistically high values of hydraulic conductivity.

It was further found that API fixed-ring hydraulic conductivity did not decrease significantly as applied pressure was increased from 2 psi to 10 psi. This trend was observed in three of the samples cited above which were initially subjected to 2 psi and subsequently increased to 3 psi, 5 psi, 7 psi, and 10 psi as well as in four other samples in which gradient was varied (refer to Table 2-3 for summary of results). The increase in applied pressure over this range appeared to have an insignificant effect on consolidation of the granular Gilson Road backfill (this conclusion might not be true for a more clayey backfill in which sample consolidation would be a larger factor). Test results indicated that increasing the pressure by 400 percent (from 2 to 10 psi) caused a decrease in hydraulic conductivity of less than 50 percent.

TABLE 2-3

EFFECT OF VARYING GRADIENT IN API FIXED-RING TEST

Station	Depth	Hydraulic Conductivity x 10^{-7} cm/sec				
		Gradient				
		28	41	69	97	138
1+00	20'-22'	0.45	0.58	0.64	0.56	0.55
1+00	41'-43'	2.1	1.4	0.82	0.67	0.52
3+45	3.5'-5.5'	1.5	1.4	1.2	1.1	0.8
3+45	11'-13'	0.88	2.0	1.1	0.99	0.97
3+45	22'-24'	0.55	0.78	0.73	0.27	--
3+45	32'-34'	1.5	1.0	0.9	0.8	0.8
11+00	9'-10.5'	3.9	3.1	2.8	2.3	2.3

2.1.3 Guidelines for Field Use of the API Fixed-Ring Test

Based on the data presented herein, and the field data from actual cutoff wall projects undertaken to date, the API fixed-ring test appears to yield reliable and accurate values of hydraulic conductivity. The reliability and accuracy combined with the relative simplicity, short testing time, and low cost of the test make it ideal for field quality control. The test can therefore be used as the primary field measure (production test) of backfill hydraulic conductivity provided accuracy is verified for each new project.

Verification testing and production testing of backfill are described in the following subsections. It will be seen that the key to verification testing is obtaining an API fixed-ring sample as representative of the corresponding triaxial sample as possible, while the key to production testing is obtaining a sample as representative of the in-place wall backfill as possible. This difference reflects the different objectives of the two series of tests: verification testing validates test accuracy; production testing validates backfill hydraulic conductivity.

2.1.3.1 Verification Testing

Verification testing should begin during the design phase of a project and continue during construction. During the design phase, a limited number of triaxial tests should be run to verify the site specific accuracy of the API fixed ring procedure. This check between API fixed-ring and triaxial data should be made after backfill component selection and proportioning are established. As part of the verification program, the driving pressure used for API fixed-ring testing should be varied to determine which value yields the best API/triaxial match. Samples selected for testing should represent the contract design mix (or mixes). Backfill samples from which aliquots will be taken for API/triaxial comparisons should be throughly mixed to be as homogeneous as possible to yield aliquots as identical as possible. Splits of a poorly mixed sample may reveal significant variations in gradation which will lead to different hydraulic conductivities. The variation in hydraulic conductivity would then be misinterpreted as testing inaccuracy.

This design testing should then be followed by triaxial testing of actual backfill samples obtained during construction. Samples selected for testing should cover the complete range of hydraulic conductivities and backfill types encountered on the project as well as represent the entire wall perimeter. To enhance the usefulness of triaxial data obtained during construction, samples should also be tested as soon as they are taken. Hence, the site specific validity of the API fixed ring data can be established well before the end of construction. The field samples from which aliquots are taken for the API/triaxial comparisons must also be as homogeneous as possible. Therefore, additional mixing far beyond that actually attained in the field is mandatory prior to testing.

2.1.3.2 Production Testing

Although a high degree of additional backfill mixing is required for the API/triaxial comparison samples, mixing of field samples taken for QC production testing in excess of that achieved during construction may yield unconservatively low values of hydraulic conductivity. Hydraulic conductivity is a function not only of the proportions in which backfill components are mixed, but also of the degree to which they are mixed. Therefore, additional mixing of a sample obtained for QC testing may improve the degree of blending to the point where the sample is no longer representative of the field mix and yields a falsely low value of hydraulic conductivity

Triaxial data from five undisturbed tube samples obtained from the completed wall indicate that up to an order of magnitude reduction in hydraulic conductivity can occur after the undisturbed sample is mixed thoroughly and retested. Results are illustrated on Figure 2-3 and summarized in Table 2-4. In each of the five cases, the remixed sample yielded a lower hydraulic conductivity than the undisturbed samples. Therefore, in some cases, the backfill may have been mixed uniformly in the field to a scale similar to that of sample size but not to the grain size scale which controls hydraulic conductivity.

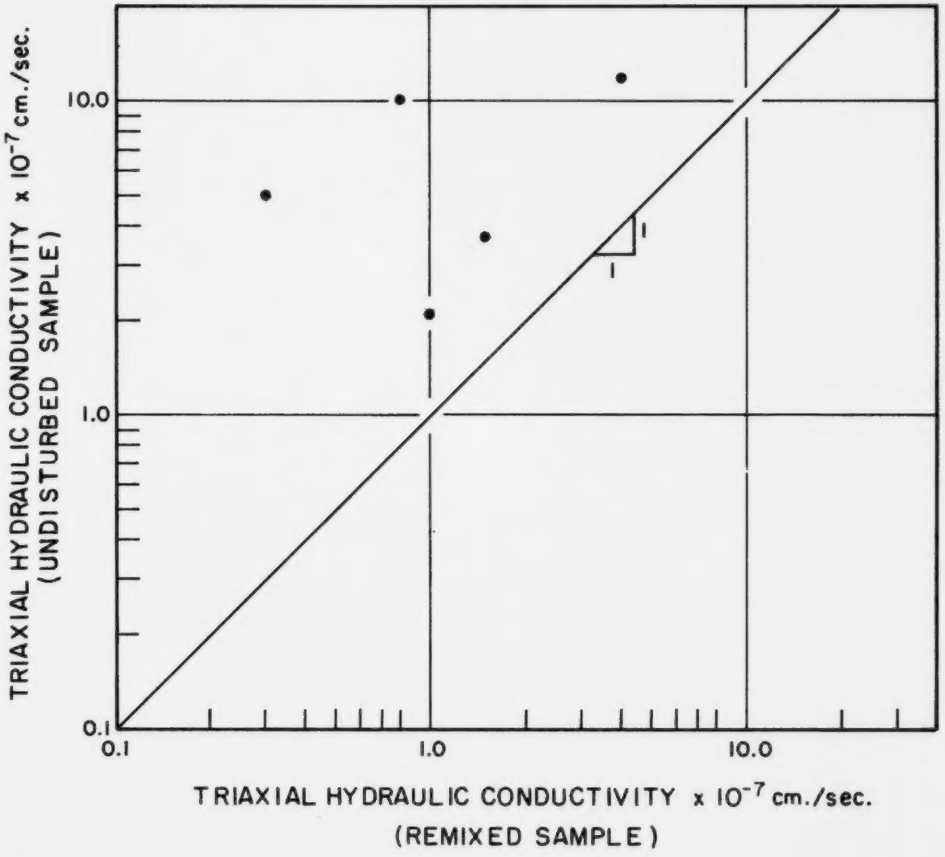
TABLE 2-4

TRIAXIAL HYDRAULIC CONDUCTIVITY:
"UNDISTURBED" VS "REMIXED" SAMPLES

Station	Depth	Hydraulic Conductivity $\times 10^{-7}$ cm/sec Undisturbed	Remixed
1+00	20-22	2.1	1.0
1+00	80-82	10.0	0.8
3+45	11-13	12.0	4.0
3+45	22-24	3.6	1.5
11+00	22-24	5.0	0.3

To obtain the most representative values of hydraulic conductivity, samples for QC production testing should be taken and placed in the test chamber with as little additional mixing as possible while still removing air voids. Failure to observe this procedural "detail" can result in artificially low (unconservative) values of hydraulic conductivity. It also

FIG. 2-3 TRIAXIAL HYDRAULIC CONDUCTIVITY RESULTS: "UNDISTURBED" vs. "REMIXED" SAMPLES



follows directly from this discussion that both the API fixed-ring and triaxial data sets obtained for verification testing do not qualify as QC production data. As indicated, the sole purpose of these tests is to demonstrate the site specific accuracy of the API fixed-ring procedure.

The driving pressure used during API fixed-ring QC testing should be that value which, based on the verification testing, yields a hydraulic conductivity closest to that derived from triaxial testing. It is emphasized that, for the reasons discussed previously, significantly higher driving pressures should not be used in an effort to increase the flow per unit time so as to make effluent volume measurements easier. Rather, smaller graduated cylinders should be employed to increase the reading accuracy for the small effluent volumes resulting from particularly low hydraulic conductivity backfills.

2.2 Evaluation of Backfill Bentonite Content Using Methylene Blue Titration

The cutoff wall at the Gilson Road site was constructed using a backfill consisting of on-site soils, off-site borrow, and bentonite. The on-site soils were highly permeable sands and gravelly sands excavated from the trench (trench spoil). Off-site borrow consisted of a quartz based fine sandy silt and was mixed with the trench spoil to reduce its hydraulic conductivity. Additional reduction of the mix hydraulic conductivity to design values ($<1 \times 10^{-7}$ cm/sec) was accomplished via bentonite addition, first as a dry powder and then as a bentonite/water slurry.

For QC of backfill proportioning and mixing in the field, testing of percent bentonite is second in importance only to measurement of the actual hydraulic conductivity. For predominantly granular backfills in particular, bentonite is the singularly most significant component responsible for reduction in hydraulic conductivity of the mix. Up to a critical level (previously determined by laboratory testing) the higher the bentonite content in the backfill, the lower and thus more desirable the overall hydraulic conductivity. For this reason, project specifications for S/B cutoff wall construction typically include a minimum requirement for percent bentonite (5% for the Gilson Road wall). In recognition of the above, the magnitude of percent bentonite in a backfill sample is the first indicator to be evaluated in the event that the API fixed-ring hydraulic conductivity testing yields unacceptable values.

In the past, field measurements of bulk quantities of backfill soils and bentonite have been used to determine bentonite percent by applying basic weight/volume relationships. This method is approximate due to the limited accuracy of the bulk weight/volume estimates, and optimistically assumes the bentonite is evenly mixed throughout the backfill. Thus, bulk measurements can be used as a gross check on the percentage of bentonite in the backfill, but are inadequate for quantitative field control.

As an alternative, the use of conventional hydrometer test procedures have been attempted for bentonite determination. This procedure is routinely used to determine the amount of silt and clay size particles in samples of natural soils. Inherent limitations in the test, however, restrict its use for backfill QC testing. Lambe (1951) reports that the hydrometer test can measure colloidal factors to an accuracy of only ±3 percent, an unacceptable error band when compared to the specified bentonite content of 5 percent. Moreover, the test cannot differentiate bentonite from other naturally occurring clay-size particles, necessitating additional hydrometer tests on the other components of the mix. Equally restrictive is the three to four days required to complete each group of tests. This time period is excessive relative to the rate at which backfill is mixed and placed. Hence, the test is considered unsuitable for real time quality control of the construction process.

A laboratory testing program was therefore carried out to investigate the applicability of the quicker and potentially more accurate methylene blue test for determination of percent bentonite in S/B backfill. A primary objective of this work was to verify the accuracy and precision of the methylene blue test procedures previously adopted for evaluation of bentonite content on the Gilson Road project. As such, primarily granular mixes in which the bentonite additive is the predominant clay component were used during the testing to simulate the backfill used at Gilson Road. In addition, evaluations of the test's applicability for general use in field control of other sites employing S/B cutoff walls were made. The test and testing program are described in the following sections.

2.2.1 Methylene Blue Titration

Methylene blue titration is a laboratory technique commonly used to determine bentonite content in bentonite/water slurries (Alther, undated). Primary applications in the past have been in the oil drilling industry for testing drilling mud. The procedure is based on the proportionality between clay content and ion exchange capacity. In a suitable aqueous environment, such as an electrolytic clay- water suspension, clay materials are associated with naturally occurring adsorbed cations of a fixed total charge (typically sodium, calcium, magnesium, or potassium). These cations are either retained by the clay or exchanged for other cations subsequently introduced into solution. The degree to which this exchange may occur, referred to as cation exchange capacity, increases with the proportion and activity of the clay in suspension.

The determination of bentonite content via the methylene blue test therefore relies on the proportionality between the amount of methylene blue cations exchanged and the amount of bentonite in the backfill sample tested. The introduction of a methylene blue solution into a bentonite-water suspension will cause such an exchange. Bentonite content can then be determined by quantifying the amount of cations exchanged. In a similar sense, bentonite content in a suspension of S/B backfill and water can also be determined using the proportionality with exchanged cations.

The quantity of methylene blue cations exchanged is determined by titration. The titration procedure involves the incremental addition of methylene blue solution to a suspension consisting of the backfill sample and a predetermined volume of distilled water and dispersent until the cation exchange capacity of the suspension is exceeded (end point). Methylene blue consumption at the end point is then used to determine the dry weight of bentonite in the sample via a pre-established correlation.

The correlation is based on a series of methylene blue tests performed on laboratory prepared samples with known concentrations of the individual backfill components (bentonite, trench spoil, and borrow). The test results indicate the cation exchange capacity of each component over the range of concentrations anticipated in the backfill. The correlation curve for the backfill is then established via the superposition of these results. As bentonite has a high cation exchange capacity relative to the granular trench spoil and borrow, cation exchange capacity of the backfill is highly dependent on bentonite content and only marginally affected by changes in spoil and borrow content.

The methylene blue test is well suited for quality control of S/B backfills due to the relatively short testing time (approximately 30 minutes per test), low cost, and minimal laboratory requirements. A detailed description of the standardized test procedure, which directly reflects the results of efforts performed for this project, is described in Appendix B.

The procedure is based on general bentonite titration methods (Alther, undated and Baroid, undated), but includes modifications necessary for specific application to quality control of S/B cutoff wall construction. These modifications affected primarily sample preparation and computational techniques.

2.2.2 Testing Program

Evaluation of the accuracy and precision of the methylene blue test for quality control of S/B backfill was separated into four distinct phases. Phasing was implemented to progressively assess the impact attributable to increasingly more complicated soil/water matrices accompanying the bentonite. These matrices encompassed the simplest case of distilled water without soil to more realistic combinations approximating actual backfill samples, including natural soils with non-bentonite related cation exchange capacity and interstitial water contaminated with hazardous wastes. The applicability of linear proportionality, matrixing effects and superposition were thus investigated. Testing accuracy was assessed through comparisons of measured bentonite content with that actually added to the laboratory prepared samples.

The four phases were:

- Bentonite in a distilled water matrix;
- Bentonite in an Ottawa sand and distilled water matrix;
- Bentonite in a contaminated water matrix;
- Bentonite in an off-site borrow, trench spoil and distilled water matrix.

The rationale leading to the selection of the above four matrices, as well as the results of the work, are presented in the following subsections.

2.2.2.1 Distilled Water Matrix

Bentonite in a matrix of distilled water and dispersant was chosen for the first phase of testing to assess the proportionality and inherent accuracy/precision of the methylene blue titration for bentonite content. This series of tests allowed evaluation of the basic test procedure without the complicating effects that additional backfill components might present.

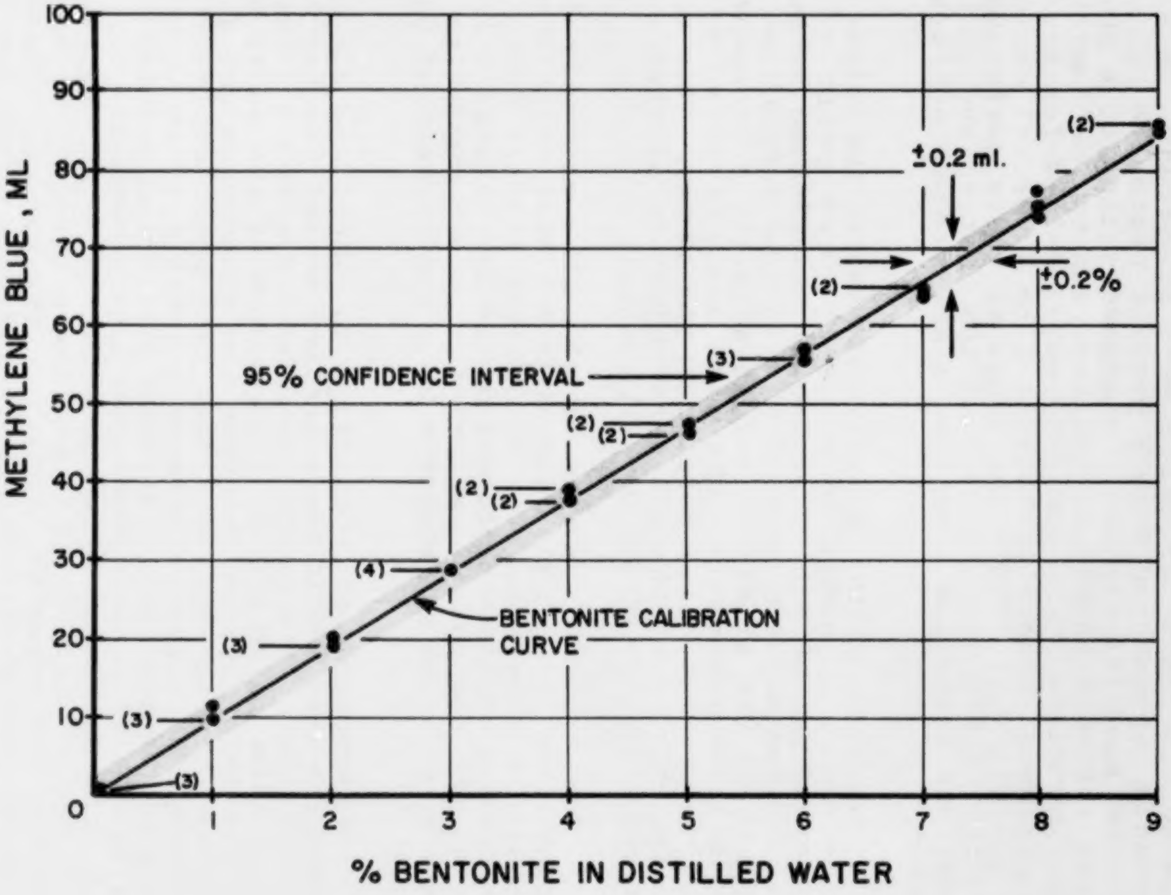
Three to four aliquots each of ten slurries containing from 0.0 to 0.9 grams bentonite were prepared. These bentonite amounts represented 0 to 9 percent bentonite by dry weight in 10-gram backfill samples. Standardization of a 10-gram sample size reflects sampling, testing, and typical backfill gradation considerations. Designed S/B backfill mixes for hazardous waste containment generally require 2 to 5 percent bentonite and as such fall well within the range of tests performed. The same bentonite was used throughout the testing and conformed to the American Petroleum Institute's API grade, minimum 91 barrel yield.

For each of the 36 individual tests, the volume of methylene blue solution added to reach the end point of titration (completion of cation exchange) was plotted versus the known percent bentonite referenced to a 10-gram sample (see Figure 2-4). (Although the titration volume actually corresponds to the dry weight of bentonite, the data are presented herein as an equivalent bentonite percent in the sample. This format is used in recognition of the ultimate objective of standardizing this procedure for QC of S/B backfill mixes).

The trend portrayed by the data not only demonstrates proportionality but also indicates a linear relationship between methylene blue titration volume and percent bentonite. Linear regression analysis performed on the data yields a line referred to as the "bentonite calibration curve" which fits the data set to a high degree as reflected by a coefficient of determination equal to 0.999.

Assumption of linearity, as supported by the physical processes involved in the titration as well as the data, allows evaluation of the test's precision and accuracy. Inasmuch as the calibration curve itself is determined by the data, accuracy and precision of the calibration are synonomous and defined as the degree to which an individual data point would be expected to correspond to the relationship defined by the entire data set (the regression line). Standard statistical methods were used to establish 95 percent confidence limits about the mean response of the data set (also shown on Figure 2-4). The resulting 0.2 percent offset of upper and lower bound error bands from the calibration curve is therefore the primary measure of both accuracy and precision. Hence, given a slurry sample containing an unknown amount of bentonite, it would be expected that the titration volume in conjunction with the previously established calibration curve would yield the correct percent bentonite to within ±0.2 percent with a 95 percent degree of confidence.

FIG. 2-4 METHYLENE BLUE TEST RESULTS: BENTONITE IN DISTILLED WATER



The bentonite/distilled water samples used to establish the basic relationship between titration volume and percent bentonite excluded all other possible backfill components. This "best case" condition is therefore expected to yield the highest degree of accuracy. The curve defined by this data is the baseline calibration against which the results of subsequent tests on more realistic backfill samples are compared and testing accuracy assessed.

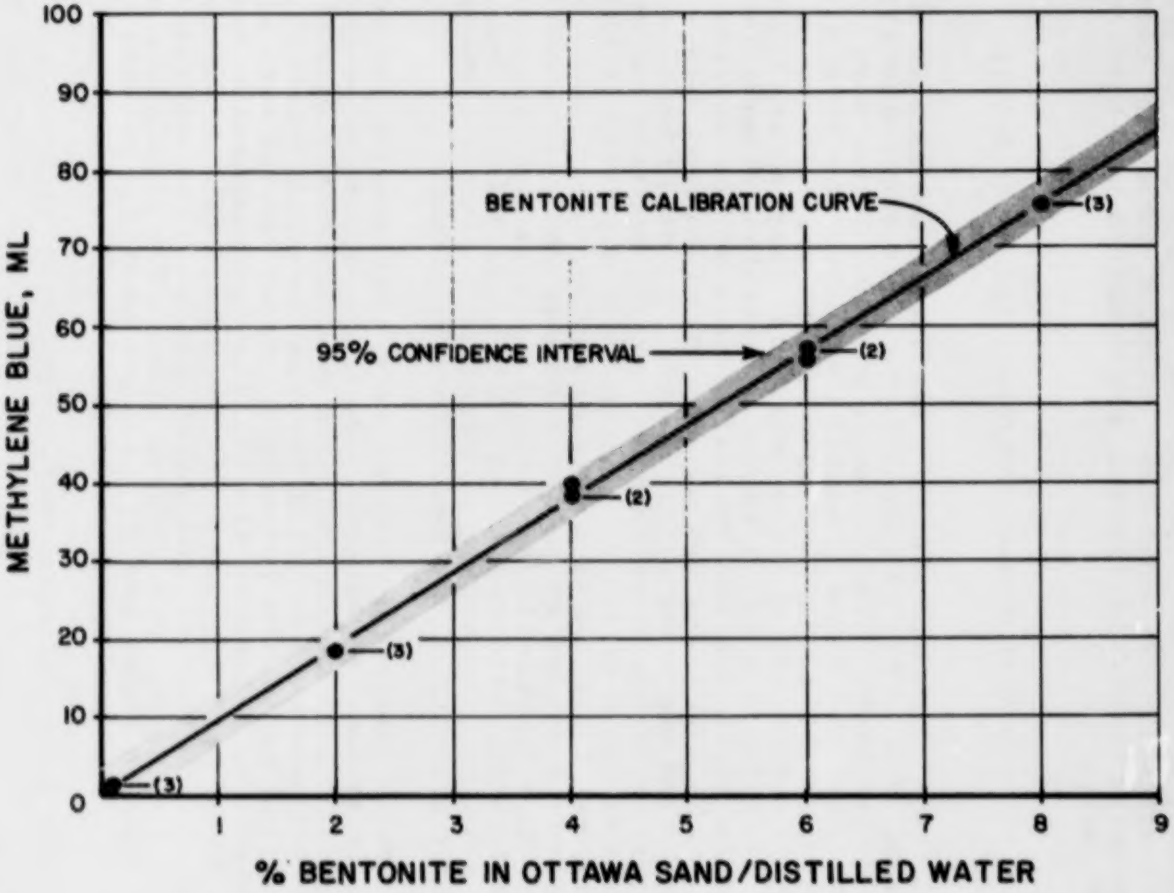
2.2.2.2 Ottawa Sand/Distilled Water Matrix

For the second phase of testing, known amounts of bentonite were added to a matrix composed of Ottawa sand, distilled water, and dispersant. These samples were one degree more complex than those used for Phase I testing and also one step closer to a typical backfill sample. Ottawa sand was selected in that it is almost entirely quartz with no clay minerals present. Furthermore, quartz is the most prevalent mineral in the Gilson Road backfill due to the silica base of the on-site soils and off-site borrow used as the predominant mix components (determined via x-ray diffraction). As would be expected based on cation exchange principles, quartz, and thus the Ottawa sand, has no exchange capacity. The specific objective of this testing phase was therefore to determine whether the presence of backfill components having no cation exchange capacity would interfere with the bentonite/methylene blue cation exchange. This interference, referred to as "matrixing effect," would include any physical, chemical, or electrical phenomena which alter the surface activity of the bentonite clay mineral.

Three identical samples were individually prepared at each of five bentonite contents: 0, 2, 4, 6, and 8 percent (by dry weight). Each sample contained appropriate dry weights of Ottawa sand and bentonite to yield 10-gram "backfill" samples with the specified bentonite percentage. These samples were added to the standardized volume of distilled water/dispersant and slurried for testing. For each of the 15 tests, the methylene blue titration volume at end point was plotted versus the known bentonite percentage (Figure 2-5). The previously determined calibration curve and 95 percent confidence interval (from Figure 2-4) are also presented with the data for comparison.

The data indicate that, within the ±0.2 percent error band established via the 95 percent confidence interval, the Ottawa sand has no effect on the methylene blue/bentonite cation exchange process. Therefore, matrixing problems are not probable due to the presence of quartz-based backfill components with no cation exchange capacity.

FIG. 2-5 METHYLENE BLUE TEST RESULTS: BENTONITE IN OTTAWA SAND/DISTILLED WATER



2.2.2.3 Contaminated Water Matrix

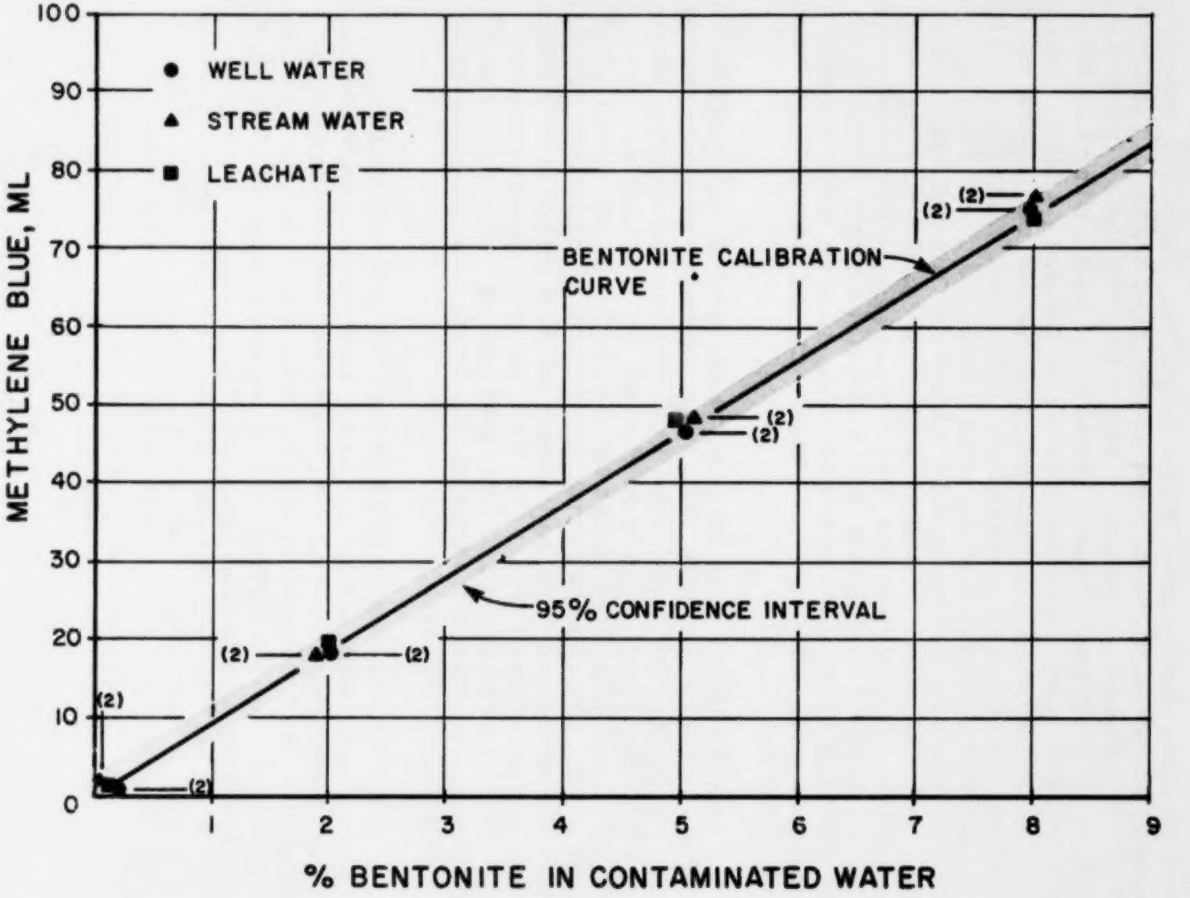
During methylene blue testing, backfill samples are, in all cases, mixed with distilled water to create a suspension which is then titrated. Hence, the water used during the testing is always contaminant free. However, contaminants may be added to the suspension via the backfill sample itself. Where the cutoff wall is excavated through contaminated portions of the aquifer, trench spoil incorporated into the backfill will contribute contaminants carried in its interstitial fluid. Furthermore, trench slurry is typically added to the backfill during mixing to increase the slump. Water added in this way typically accounts for over 50 percent of the backfill water content. While the slurry may also pick up contaminants from the aquifer's interstitial fluid via osmosis and/or entrainment, of greater importance is the chemical make-up of the water used to produce the slurry.

In light of the above, addition of contaminants to the test suspension may be more the norm rather than the exception. This was in fact the case for the Gilson Road site where the primary aquifer contaminants consisted of volatile organic industrial solvents, and the slurry makeup water exhibited significant hardness. To assess the possible matrixing effects of these contaminants on the accuracy of the methylene blue test, a worst case scenario was utilized for sample preparation. In addition to incorporation of contaminants via the backfill itself, contaminated water was used to form the test suspension instead of distilled water.

Contaminated water from three different sources on the Gilson Road site were obtained for this phase of the work. The first two, designated well water and stream water, were sources actually used during construction for slurry makeup water. The third and most contaminated source was leachate as taken from monitoring installations on the site. Backfill samples were prepared at 0, 2, 5, and 8 percent bentonite in a manner similar to Phase I testing. Two aliquots were tested for each bentonite content using the stream and well water (total of 16 samples). Only one series was performed using leachate at the four bentonite contents (4 samples).

Test results, presented in Figure 2-6, indicate that the contaminants tested had no significant effect on the test results. All but one of the individual data points fall within ±0.2 percent of the original bentonite calibration curve. As such, the initial calibration accuracy of ±0.2 percent with a 95 percent confidence level was maintained in spite of the Gilson Road contaminants introduced into the matrix tested.

FIG. 2-6 METHYLENE BLUE TEST RESULTS: BENTONITE IN CONTAMINATED WATER



For volatile organic contaminants, noninterference would be expected inasmuch as they are non-ionic and therefore should not exhibit a high cation exchange capacity. In addition, the backfill sample is dried in an oven at 105°C and the backfill/water/dispersant matrix is boiled prior to titration. The temperature involved in the above procedural steps would be expected to remove the volatiles.

For other sites, particularly those containing highly ionic contaminants, the cation exchange potential of the species present should be compared to that of methylene blue with respect to affinity for the bentonite clay mineral. In cases where high non-volatile solvent concentrations exist, reduced solvent (water) polarity effects, as they relate to methylene blue solubility, should also be investigated. This can be done on a theoretical basis or by utilizing the simple testing procedure described herein. It should be recognized however that this testing procedure can yield "false positives" in that the quantity of contaminants introduced into the matrix far exceeds that which is likely to be added via an actual backfill sample.

2.2.2.4 Gilson Road Backfill Matrix

The fourth phase of testing employed prepared samples duplicating the backfill used for the Gilson Road project. As previously discussed, the Gilson Road backfill consisted of approximately equal proportions of trench spoil (sand and gravel) and off-site borrow (sandy silt) with 3 to 5 percent bentonite added. The objectives of this series of tests were to 1) evaluate the impact of a cation exchange capacity attributable to non-bentonite backfill components on the determination of percent bentonite in the backfill; 2) determine if the bentonite calibration could be corrected to account for non-bentonite cation exchange capacity utilizing superposition; and 3) based on the above, evaluate the accuracy of the methylene blue test results previously obtained for the Gilson Road project.

The on-site soils (trench spoil component of the backfill) used for testing were obtained during a post-construction boring program. Samples were selected from various locations, both with respect to wall perimeter and aquifer depth. They exhibited gradations ranging from fine sand and silt to fine to coarse sand and gravel and represent the full variation of soils encountered during cutoff construction. The ten samples were mixed with distilled water and dispersant in a manner similar to Phase II testing except that no bentonite was added. The on-site soils exhibited average cation exchange capacities equivalent to 0.2

percent bentonite with a range of 0.1 to 0.4 percent. Results are summarized in Table 2-5.

TABLE 2-5

METHYLENE BLUE TEST RESULTS
ON-SITE (TRENCH SPOIL)

Station	Offset	Depth Feet	Description	Methylene Blue, Ml
2+50	20'S	0-2	fine sand and silt	4
9+30	50'N	33-37	fine sand	3
9+30	50'N	11-13	fine to coarse sand	1
13+00	0	24-26	fine to medium sand	2
13+00	0	44-46	fine to coarse sand	4
34+70	60'NW	54-55.5	glacial till	1
34+70	60'NW	79-80.5	glacial till	1
25+30	20'NE	29-31	fine to coarse sand and gravel	2
25+30	20'NE	69-71	fine to medium sand	2
38+50	40'E	24-26	fine to coarse sand sand gravel	3

Nine samples of the off-site borrow were also selected from record samples taken during construction. Again the objective of sample selection was to encompass the full variability of the materials. Eight of these samples yielded bentonite equivalents of 0.6 to 0.7 percent with one sample yielding 0.4 percent. Test results are summarized in Table 2-6.

TABLE 2-6

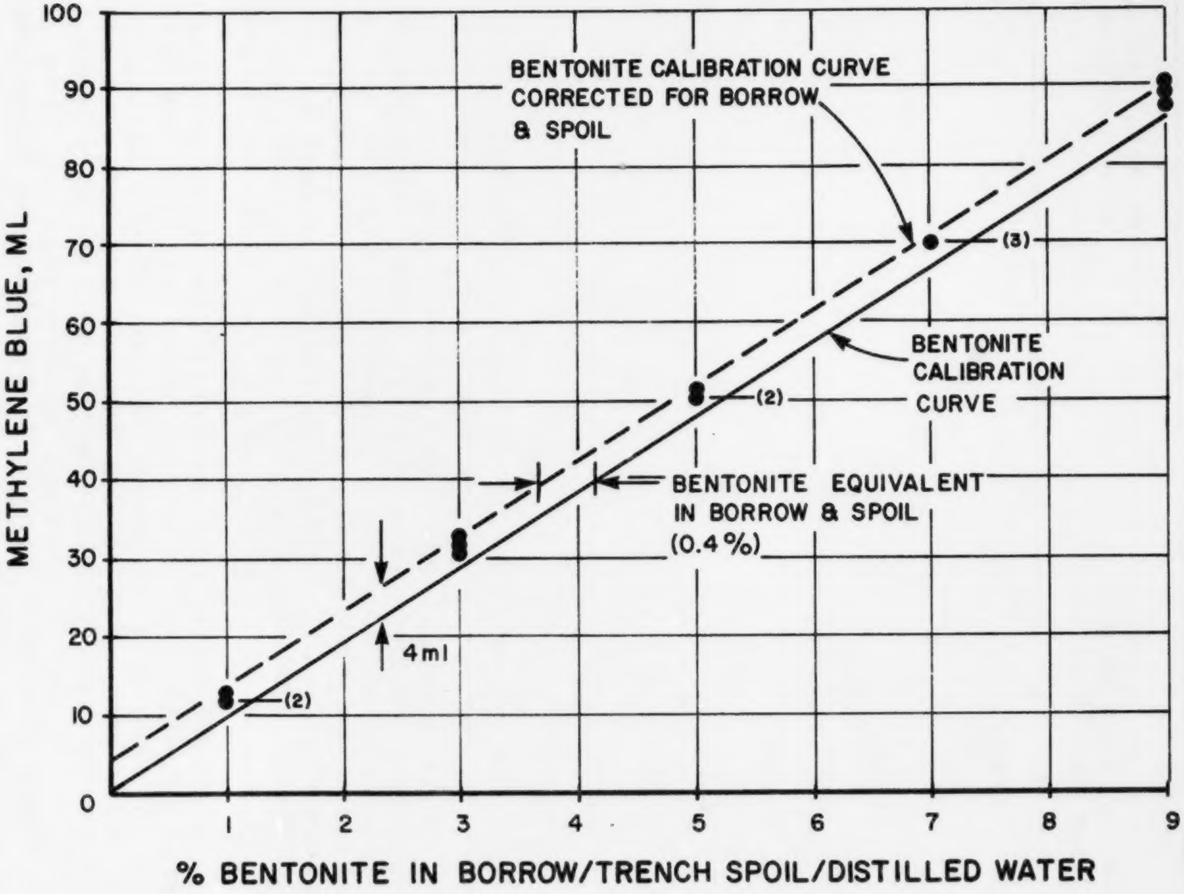
METHYLENE BLUE TEST RESULTS
OFF-SITE BORROW

Station From Which Borrow Was Obtained	Millileters of Methylene Blue Required to Complete Titration
4+45	7
9+00	7
12+50	6
18+00	7
25+50	4
28+00	6
32+00	7
35+00	6
38+50	6

This preliminary testing for the exchange capacity of individual non-bentonite backfill components provided a basis for prediction of the bias expected relative to the bentonite calibration previously developed. An anticipated offset equivalent to 0.4 percent bentonite was derived using a weighted average of the individual exchange capacities based on the relative amounts of borrow and trench spoil in the backfill.

Fifteen splits of backfill were then prepared containing equal proportions of the on-site soil and the off-site borrow (ratio used during construction of the Gilson Road cutoff). Bentonite was added to the resulting mixture (a silty sand and gravel) to obtain three samples at each of 5 bentonite contents: 1, 3, 5, 7, and 9 percent by dry weight. The samples were titrated with methylene blue solution until endpoint; the data are plotted on Figure 2-7.

FIG. 2-7 METHYLENE BLUE TEST RESULTS:
BENTONITE IN BORROW/TRENCH SPOIL/DISTILLED WATER



Test results exhibited a linear trend, but, as expected, also demonstrated bias reflecting greater exchange capacity than attributable to the bentonite alone. Superposition of the previously predicted 0.4 percent non-bentonite derived off-set on the original calibration curve yielded a "corrected calibration curve" presented as a dashed line on Figure 2-7. The original bentonite calibration curve developed during the first phase of testing is also presented for comparison.

Although this corrected calibration does not exactly match the best fit line through the actual data, it does agree to ±0.3 percent bentonite. These data therefore support the validity of superposition in that the small discrepancy obtained corresponds to inaccuracies incurred by averaging the data upon which the corrected calibration curve was based. These results indicate that the procedure of superimposing average exchange capacities of non-bentonite backfill components on the bentonite calibration curve, as was used on the Gilson Road project, yields a new corrected calibration curve with an expected accuracy of ±0.3 percent bentonite. This error band represents less than ±6 percent of the specified bentonite content (5 percent of backfill, by dry weight) and verifies that the data amassed during the cutoff wall construction at Gilson Road were in fact sufficiently accurate for field QC use.

It should be emphasized at this point that unless the bentonite calibration curve is corrected for the cation exchange capacity attributable to non-bentonite backfill components, the bentonite contents measured during QC testing will be unconservatively high. In addition, the accuracy of this superposition-based correction is directly dependent on the uniformity of the cation exchange capacities of the individual components and the proportioning of these components in the backfill mix. Therefore, if either of these parameters are varied during construction of the cutoff, then separate calibration curve corrections representing individual sections of the wall would be required for QC testing.

2.2.3 Field Use of the Methylene Blue Test

The laboratory program described herein was used to assess the accuracy of the methylene blue test and was therefore structured to measure the unknown quantity of methylene blue required to titrate backfill samples with known bentonite contents. The data thus developed demonstrated the accuracy of the test for field QC in cases similar to the Gilson Road project. During actual use for QC in the field, however, the methylene blue consumption would be measured and used to determine the unknown bentonite content in the backfill via pre-established calibrations. Thus, the testing methodology used in field applications initially would follow a logic similar to the laboratory program for establishment of a calibration curve, but thereafter utilizes methylene blue consumption as the independent variable and bentonite content as the dependent variable.

Implementation of the methylene blue test for quality control therefore consists of the following procedure:

- Establish a bentonite calibration curve representing the relationship between methylene blue consumption and bentonite content for bentonite/distilled water suspensions. The same bentonite to be used during construction must be used during this step. The procedure would be similar to the Phase I testing described herein except that the methylene blue consumption would be plotted on the x-axis as the independent variable.

- Determine whether contaminants are likely to be present in the backfill samples and evaluate the probable effect on the calibration.

- Determine average cation exchange capacity (methylene blue consumption) of the non-bentonite backfill soils combined in the same proportions they will be mixed in the field. Use the bentonite calibration curve to determine the bentonite equivalent for this volume.

- Offset the initially determined bentonite calibration curve to account for the bentonite equivalent of the non-bentonite soils and/or contaminants to establish a field calibration curve. Note that multiple field calibration curves may be required if significant variation in the offset values are anticipated due to variations in the site geology or construction procedures.

- Perform the methylene blue test on backfill samples and enter the field calibration curve with the volume of methylene blue consumed at the end point to determine bentonite content.

It should be pointed out that although the test is quick, inexpensive, and accurate, an initial level of practice is required to accurately assess the end point of titration. Therefore, only technicians having experience with the titration procedure should be allowed to perform the tests under the rigors imposed by field quality control of fast track construction.

Finally, the procedures presented herein are not directly applicable to predominantly clay-based backfills. For this class of cutoff, the high proportion of non-bentonite clay components and the variability of their total percentage in the backfill can make correction of the bentonite calibration curve "sample specific." Therefore, at the very least, independent knowledge of the amount of non-bentonite clay in the sample would be required. A procedure has been developed for this case allowing determination of bentonite content through solution of simultaneous equations. This work will be presented in a future report once the precision and accuracy of the procedure have been established.

2.3 Effect of Backfill Gradation on Hydraulic Conductivity

As discussed in the previous sections, the addition of relatively small percentages of bentonite to a granular soil will reduce its hydraulic conductivity several orders of magnitude. This effect is widely recognized and has been clearly demonstrated in laboratory tests (Schulze, 1984) and field applications. It is also recognized, though less well understood, that the overall gradation of a backfill sample also effects its hydraulic conductivity. Inherently, the more "well-graded" the sample, the denser the particle packing and hence, the lower the hydraulic conductivity. It is not clear, however, what effect various gradation parameters have on hydraulic conductivity.

As part of this project, therefore, backfill data collected both during and after construction were statistically analyzed to determine to what extent gradation characteristics of a backfill sample could be used to predict hydraulic conductivity. The ultimate objective of this effort was to identify which aspects of backfill gradation play the greatest role in determining backfill hydraulic conductivity. This information would then serve to enhance quality control procedures by providing additional insight into a backfill mix which API fixed-ring testing indicates has an unacceptably high hydraulic conductivity. With such a correlation, gradation analyses of backfill could become a more integral part of the quality control program. At the present, gradation parameters are typically used only to control the percentage of fines within a minimum and maximum value. No further criteria to control, much less optimize, backfill gradation are included in specifications because the influence of varying gradation characteristics is not sufficiently understood.

The method used to evaluate the correlation consisted of the following:

- Establish a data base of gradation parameters for a statistically meaningful number of backfill samples. For each of these samples, determine hydraulic conductivity.
- Perform linear regression analyses on the individual parameters to establish the degree of dependency of hydraulic conductivity on each.
- Evaluate the interaction of the individual parameters using multiple polynomial regression analyses.
- Establish confidence envelopes for the relation between hydraulic conductivity and the soil parameters.

This procedure and the results of the analyses are described in the following sections.

2.3.1 Data Base

A body of gradation parameters and hydraulic conductivity data was collected for eighty-six backfill samples obtained from the Gilson Road site both during and after construction. The primary backfill parameters for use in the correlation were percent bentonite, percent clay sizes, and percent silt sizes since hydraulic conductivity is considered most dependent upon percentage and nature of fines (material passing no. 200 sieve). Additional gradation parameters were included to describe the overall distribution of particle sizes and the grain size curve. Several density parameters were also included since hydraulic conductivity is seen as a secondary function of backfill density.

The backfill gradation and density parameters used in the analyses are listed below:

- Percentage of material passing the No. 4 sieve (by dry weight).
- Percentage of material passing the No. 200 sieve, i.e., silt and clay sizes (by dry weight).
- Percentage of material finer than 2 microns, i.e., clay sizes (by dry weight).

- Percentage of bentonite (by dry weight; determined using methylene blue titration).
- Grain size at which 60 percent of the sample, by dry weight, is finer (D60).
- Uniformity coefficient (D60/D10).
- Coefficient of curvature (D30)2/(D/10) (D60).
- Unit Weight.
- Water content (wt. water/wt. dry soil).
- Void ratio (volume of voids/volume of soil).

Values of hydraulic conductivity used in the analysis were determined by API fixed-ring testing.

2.3.2 Statistical Evaluation

As the second step in the development of a correlation, the dependence of hydraulic conductivity on each backfill parameter was evaluated. Using standard statistical methods, regression lines defining the mean response of each gradation/density parameter and hydraulic conductivity were determined. For each set of data, the correlation coefficient describing the scatter of the data about the regression line was also calculated as an index of the degree of dependency. With a correlation coefficient of 1.0 (plus or minus) indicative of perfect correlation (i.e., all data points located on regression line), a correlation coefficient of 0.75 was initially chosen to distinguish those backfill parameters having "primary" influence from those having "secondary" influence, for later use in performing polynomial regression analyses.

Correlation coefficients were first calculated based on an arithmetic scale plotting of hydraulic conductivity versus each soil parameter. Results are tabulated in Table 2-7. None of the soil parameters produced a correlation coefficient greater than 0.75; the highest coefficient was 0.42. The correlation coefficient between percent bentonite and hydraulic conductivity was 0.36; the correlation coefficient for percent clay sizes was also 0.36. For total percent fines, the coefficient was 0.27. Thus, the parameters thought to be most important to hydraulic conductivity did not emerge as such in the statistical evaluation.

TABLE 2-7

CORRELATION COEFFICIENTS
HYDRAULIC CONDUCTIVITY VS. GRADATION PARAMETERS

Y = CON-API	X vs. Y	X vs. LOG(Y)	LOG(X) vs. Y	LOG(X) vs. LOG(Y)
X = %-No.4	0.048	0.095	0.049	0.097
X = %-No.200	-0.274	-0.240	-0.309	-0.269
X = %-2u	-0.355	-0.420	-0.362	-0.418
X = D10	0.187	0.231	0.376	0.438
X = D30	0.422	0.388	0.387	0.382
X = D60	0.141	0.089	0.130	0.091
X = Cu	-0.314	-0.393	-0.300	-0.362
X = Cz	0.047	-0.004	0.094	0.074
X = GAMMA T	0.311	0.352	0.310	0.353
X = W	-0.347	-0.380	-0.351	-0.376
X = e	-0.363	-0.399	-0.368	-0.395
X = % Bent	-0.359	-0.440	-0.400	-0.471
X = CON-API	1.000	0.894	0.894	1.000

NOTES:

CON-API = API fixed-ring hydraulic conductivity.
%-No.4 = Percentage of material passing the No. 4 sieve (by dry weight); similar for "%-No.200" and "%-2u".
D10 = Grain size at which 10 percent of the sample (by dry weight) is finer; similar for "D30" and "D60".
Cu = D_{60}/D_{10}
$C_2 = (D_{30})^2/(D_{10})(D_{60})$
GAMMA T = Unit weight.
W = Water Content.
e = Void ratio.
% Bent = Percent bentonite determined using methylene blue test.

Correlation coefficients were then calculated based on combined arithmetic-logarithmic ("semi-log") scale plottings and on a logarithmic ("log-log") scale plotting to seek a better correlation. None of these relationships exhibited a correlation coefficient in excess of 0.5. Results are also summarized in Table 2-7.

As none of the individual parameters was statistically found to be a primary determinant of hydraulic conductivity, additional statistical evaluation to determine interaction of individual parameters and then confidence envelopes was not pursued. The lack of a predominant dependency of hydraulic conductivity on percent bentonite, percent clay sizes or percent fines and lesser dependency on other parameters indicated that the overall correlation between hydraulic conductivity and soil gradation parameters may be extremely complex and might not be fully reflected by the available data set. This may be due to the variable degree of blending of backfill components in the samples tested. Increased blending of field samples was found to decrease hydraulic conductivity (see Section 2.1.3.2), but it is not known to what degree the field samples were homogeneous on a grain-size scale. In addition, the correlation is affected by the previously observed variations in hydraulic conductivity measurements of identical backfill samples. The worst case precision of the test was found to be ±57 percent within one standard deviation of the mean value; this degree of variation would partially mask the existence of a close correlation.

The poor statistical correlation does not, however, alter or invalidate the premise that hydraulic conductivity is largely determined by percent bentonite or percent clay sizes. Experience has demonstrated the key roles played by these factors. Therefore, it is simply concluded that the underlying correlation between the key soil parameters and hydraulic conductivity may have been obscured by the inherent precision of the API fixed-ring test.

2.3.3 Importance of Statistical Correlation

The establishment of a statistical correlation between gradation parameters and hydraulic conductivity was investigated primarily to gain further insight into the relative importance of the various parameters on backfill hydraulic conductivity. Better correlations would have rendered gradation analyses a more integral component of field quality control testing than at the present. As a supplement to methylene blue testing, gradation analyses might have provided several key indicators of potential deficiencies in a backfill sample having an unacceptably high hydraulic conductivity.

In the absence of such a correlation, however, quality control capabilities are not seriously impaired. API fixed-ring and methylene blue testing remain the primary and secondary quality control tests. Both tests are more direct indicators of backfill quality than gradation analyses and both are quicker to run.

The greater value in establishing a statistical correlation between quality control and hydraulic conductivity is for backfill design. If the effect of various gradation parameters on hydraulic conductivity could be quantified, preliminary backfill design could consist of assessing gradation characteristics of on-site soils and determining via the correlation the various changes in gradation required to achieve desired hydraulic conductivity.

2.4 Assessment of Gilson Road QC Data

The use of the API fixed-ring and methylene blue tests during construction of the Gilson Road cutoff wall reflected the current state of the art in field quality control procedures. Although the precision and accuracy of these tests were not assessed at the time of construction, the tests were used because of their respective advantages over alternative test procedures. Moreover, the data generated during construction ultimately played a vital role in the construction control program. Tests were run on a daily basis to assess the success of the contractor in attaining design standards; deficiencies thus identified were brought to the contractor's attention and immediate corrective measures were typically taken to remedy the problem. It should be emphasized, however, that evaluation of the overall quality of the final product at the end of construction was based to a large degree on test procedures for which reliability had not yet been determined. As a result, in spite of the state of the art field control program, the degree to which the quality control data represented the quality of wall construction, and therefore, the degree to which the completed wall satisfied project specifications, was not known.

The present effort intended to achieve two major objectives: 1) assess the reliability of QC procedures used at Gilson Road; and 2) evaluate the data from wall construction in the light of this new information. In previous sections, both the API fixed-ring test and methylene blue test were found to be sufficiently accurate and repeatable to validate their use at Gilson Road. In this section, the QC data generated during and after construction are reviewed to assess the degree to which project specifications were satisfied. Where specifications were not satisfied, the impact on cutoff wall performance has been evaluated. Further, the rationale for accepting backfill which does not meet specifications is discussed.

2.4.1 Review of QC Data

Effectively two bodies of QC data were collected at Gilson Road. The first set represented tests performed on "grab" samples obtained during construction. The second set consisted of tests performed on "undisturbed" tube samples obtained from the completed wall one to three years after construction. The first set of data indicated that the hydraulic conductivity specification was generally satisfied once initial "Contractor Learning Curve" problems with backfill proportioning and mixing were overcome. The second set of data, however, indicated that the in-situ hydraulic conductivity was potentially an order of magnitude higher than that determined on the "grab" samples. The results of the QC testing are described below and evaluated with respect to specification requirements.

2.4.1.1 QC Data From Construction Samples

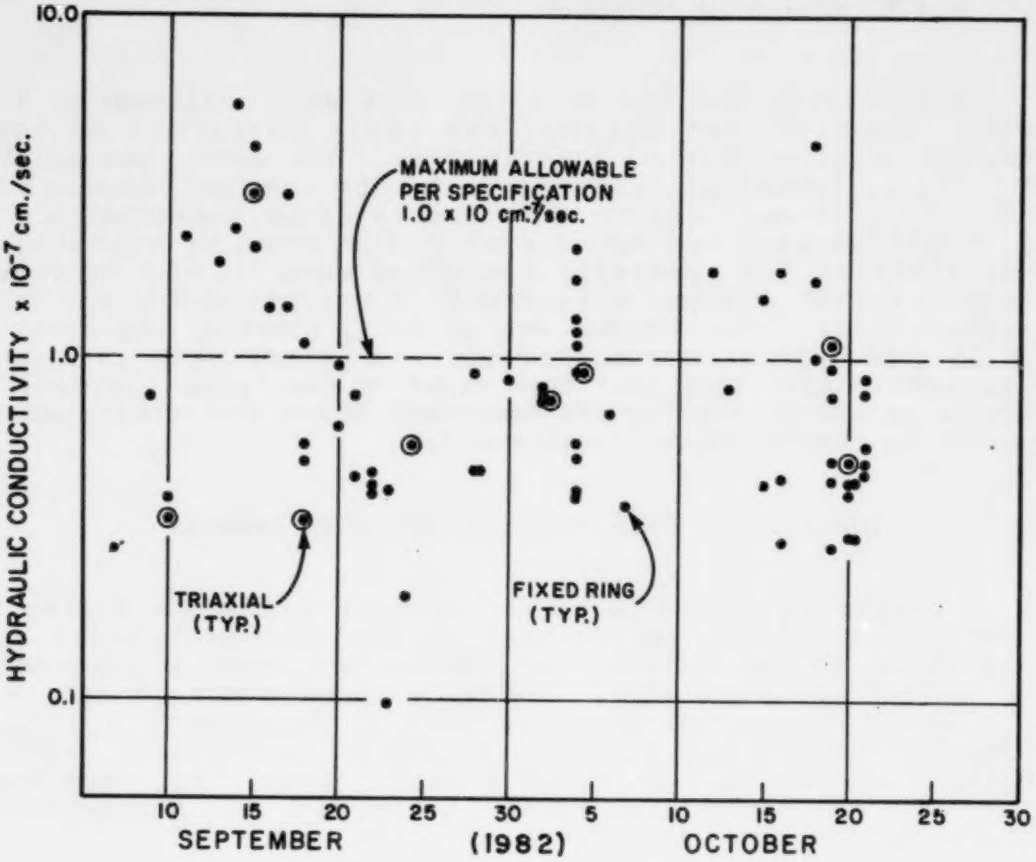
Seventy-five backfill samples were collected during wall construction. The samples were collected immediately after completion of the field mixing effort, just prior to placement of the backfill in the trench. Typically, each sample consisted of approximately 25 pounds of backfill to provide splits for hydraulic conductivity testing, methylene blue testing, gradation testing, and record samples. Prior to splitting, each sample received additional mixing by hand to improve the homogeneity of the mix and thus the similarity of splits.

API fixed-ring hydraulic conductivity tests were run on each of the seventy-five samples. In addition, triaxial tests were run on ten of the samples.

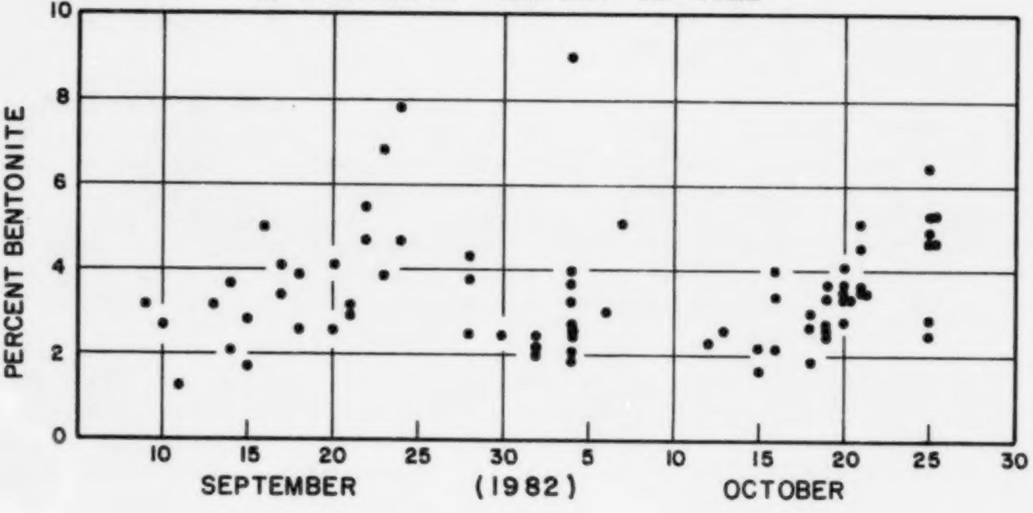
The results of the eighty-five tests ranged from 9.9×10^{-9} cm/sec to 4.3×10^{-7} cm/sec. The mean value was 5×10^{-8} cm/sec. Sixty-six tests (78 percent) satisfied project specifications for hydraulic conductivity, yielding values less than or equal to 1.0×10^{-7} cm/sec. Of the nineteen samples that yielded hydraulic conductivities greater than the specified maximum value, eleven were between 1.0×10^{-7} and 2.0×10^{-7} cm/sec, eight were between 2.0×10^{-7} cm/sec and 4.3×10^{-7} cm/sec.

A chronological plot of hydraulic conductivity versus construction date (Figure 2.8a) reveals that six of the eight highest values were recorded during the initial days of construction (9/11/82 - 9/17/82). Throughout the remainder of

FIG. 2-8 a. HYDRAULIC CONDUCTIVITY vs. TIME



b. BENTONITE CONTENT vs. TIME



the project, values in excess of the specification were encountered on five construction days spaced over the first three weeks in October. A plot of hydraulic conductivity versus station along the wall alignment (Figure 2-9a) indicates that the higher values were found between stations 0+00 and 3+00, between stations 6+00 and 12+00, and between stations 25+00 and 29+00.

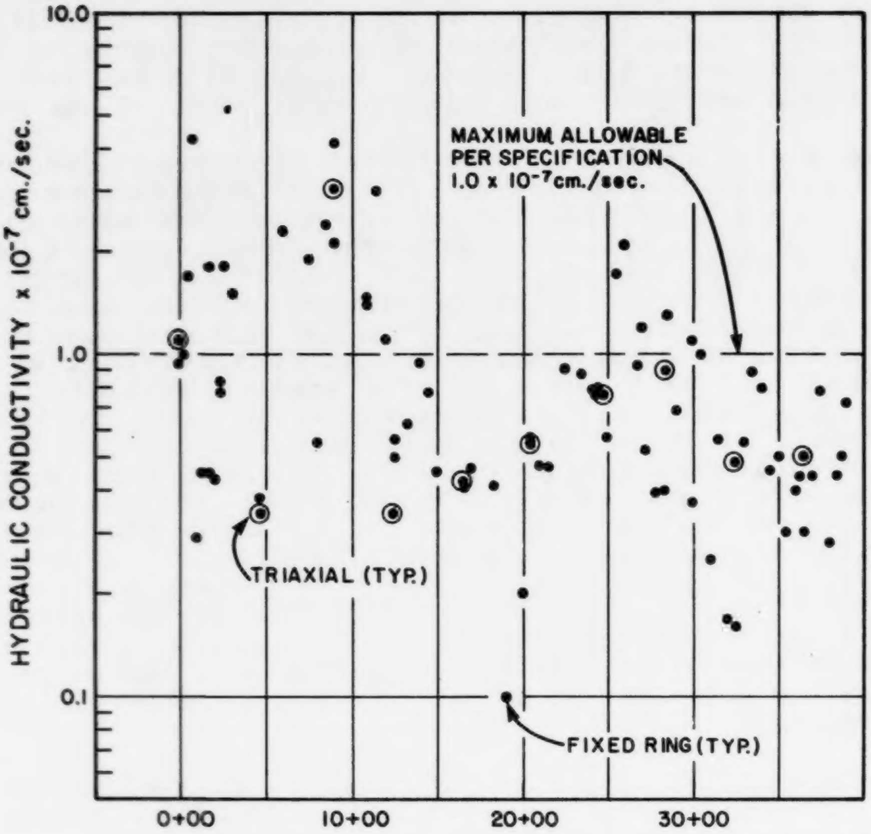
The primary cause of the higher hydraulic conductivities measured during construction appears to be insufficient bentonite content. On a time plot illustrating percent bentonite versus construction date (Figure 2-8b), the lowest percent bentonite (1.3 percent) was recorded on 9/11/82 (start of construction). The time plot further illustrates that percent bentonite was gradually increased over the first few weeks of construction to values as high as 7 and 8 percent bentonite before leveling off at a range of 2 to 5 percent. The gradual increase in percent bentonite at the beginning of construction reflects the contractor's response to the initially low values. These low values resulted from "Learning Curve Problems"[1] associated with the inaccuracies of bentonite determination via bulk measurements. The increase in percent bentonite from approximately 9/11/85 to 9/24/85 is directly reflected in a steady decrease in hydraulic conductivity over the same period (refer to Figure 2-8a). Beyond 9/24/85, the occurrence of the lowest values of percent bentonite occurred between 10/2 and 10/4 and between 10/12 and 10/18, coinciding with the occurrence (in both time and location) of hydraulic conductivities in excess of specifications.

A secondary cause of the high values of hydraulic conductivity, particularly during early stages of construction, may have been inadequate mixing effort. While bentonite contents of 2 percent or lower were measured during three periods throughout construction, high values of backfill hydraulic conductivity were most frequently measured during the initial days of construction. This situation is also reflective of a learning period that construction personnel typically experience during initial days of backfill preparation as they adjust to site specific conditions.

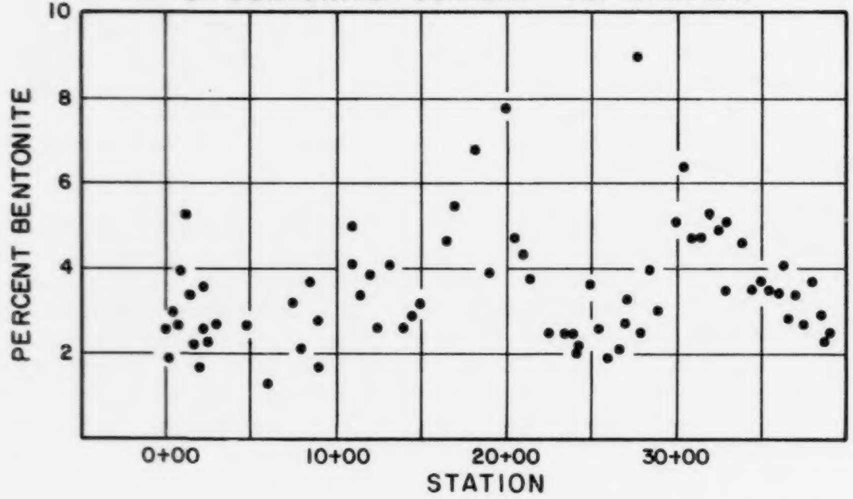
---

[1] As cutoff wall construction is a specialty contracting job, commonly executed by local equipment operators working under the supervision of two or three experienced cutoff wall personnel, a certain period of time is required for "operator education" at the initial stage of construction. The "learning curve" phenomenon can result in erratic quality for the first several hundred feet of a wall. Therefore, during initial project start-up, additional conservatism with respect to bentonite content and mixing effort should be executed.

FIG. 2-9 a. HYDRAULIC CONDUCTIVITY vs. STATION



b. BENTONITE CONTENT vs. STATION



The results of the hydraulic conductivity and methylene blue tests run on grab samples thus indicated that, in general, the hydraulic conductivity specification was satisfied while the bentonite specification was not. Although hydraulic conductivity data indicated that approximately 22 percent of the samples tested had a hydraulic conductivity in excess of the maximum specified value of 1.0×10^{-7} cm/sec, the higher values were judged to have little impact on wall performance. The impact of lower than specified bentonite values was also limited as the specified value of 5.0 percent bentonite incorporated a safety factor of 1.67 over the value required based on design testing (3.0). This factor of safety was adopted to account for the less than "laboratory perfect" proportioning and mixing expected during actual construction. Therefore, the mean value of 3.5 value reflected a technically acceptable level.

2.4.1.2 QC Data From Post-Construction Samples

Eleven backfill samples were obtained in 1983 and 1985 from the completed wall. The samples were obtained from various depths at three stations along the wall alignment, station 1+00, station 3+45, and station 11+00. These locations were chosen on the basis of the relatively high hydraulic conductivities measured on backfill samples obtained from these areas during construction (refer to Figure 2-9a). It was intended that the post-construction samples would represent "worst case" backfill. The samples were obtained using a stationary-piston tube sampler, commonly used to obtain "undisturbed" samples of fine-grained soils. Sample depths ranged from 2 to 82 feet. The tubes were x-rayed prior to sample extrusion to determine the relative gradation of the backfill samples (predominantly coarse versus fine) as well as the location of gravel pieces within the tube. The primary objective was to test a variety of backfill samples while at the same time avoiding specimens with large pieces of gravel. A large piece of gravel would result in an unrepresentatively low hydraulic conductivity as it would block off a large percentage of the cross-sectional area of the sample and the hydraulic conductivity of the gravel itself is nearly zero. The total area is used in the hydraulic conductivity computation.

Each of the eleven samples was tested "undisturbed" in the triaxial apparatus immediately upon extrusion from the sampling tube. The tests were first run at low confining pressures commensurate with top of wall conditions although eight of the samples were taken from depths below 10 feet. "Top of wall" stresses were selected to correlate with the stress range used for API fixed-ring testing executed during construction.

Quality control testing during construction was conducted using "top of wall" stresses inasmuch as the final vertical location of backfill in the wall cannot be predicted when the sample is obtained prior to backfill placement (see Section 2.1.2.2).

It was recognized that the undisturbed samples taken from the completed wall would have undergone consolidation commensurate with depth in the wall. Therefore, the soil fabric of these samples would correspond to the API fixed-ring samples only to the extent that the sample swelled in response to stress relief. The samples would thus be tested in an overconsolidated state. The significance of overconsolidation is discussed below.

Seven of the eleven samples were subsequently tested at a second, higher stress level, commensurate with the approximate vertical effective stress at the depth from which the sample was taken. This higher stress level yields hydraulic conductivities representative of the actual performance of the wall. The lower stress levels would be expected to yield higher and thus conservative hydraulic conductivity values.

The eleven tests at low stresses yielded hydraulic conductivities ranging from 6.6×10^{-8} to 1.2×10^{-6} cm/sec (refer to Table 2-8). The mean value was 5.6×10^{-7} cm/sec. Only two of the eleven tests satisfied project specifications for hydraulic conductivity. However, seven of the tests yielded hydraulic conductivities less than 5.5×10^{-7} cm/sec. Thus, as hydraulic conductivities of the construction samples taken from these areas ranged from 9.9×10^{-9} to 4.3×10^{-7} cm/sec, these "undisturbed" triaxial results appear consistent with construction results. The remaining four test results are notably higher, exceeding the previous maximum by half an order of magnitude and the specified limit by one order of magnitude.

TABLE 2-8

HYDRAULIC CONDUCTIVITY: "UNDISTURBED" VS "REMIXED" SAMPLES

Station	Depth (ft.)	Triaxial Hydraulic Conductivity, K (x 10^{-7} cm/sec)						API Fixed-Ring Hydraulic Conductivity (x 10^{-7} cm/sec)
		Undisturbed low stress		Undisturbed high stress		Remixed low stress		
		k	σ	k	σ	k	σ	
1+00	9.5-9.9	0.66	5	0.47	10	—	—	0.28
1+00	20.6-21.1	2.1	5	1.3	20	1.0	5	0.78
1+00	31.0-31.4	1.5	5	—	—	—	—	1.0
1+00	42.5-42.9	0.85	5	—	—	—	—	0.82
1+00	80.6-81.0	10.0	3	3.8	37	0.8	3	0.8
3+45	4.5-5.0	12.0	5	—	—	—	—	1.2
3+45	12.5-12.9	12.0	5	7.8	10	4.0	5	1.1
3+45	22.4-22.8	3.6	5	2.5	20	1.5	5	0.78
3+45	33.0-33.5	5.5	5	1.7	40	—	—	0.90
11+00	9.2-9.7	10.0	5	—	—	—	—	2.8
11+00	22.4-22.9	5.0	3	2.5	13	0.26	3	0.7

σ = effective confining pressure (psi).

When seven of the tests were further consolidated to approximate in-situ stress levels, the result in each case was a reduction in hydraulic conductivity. Within this subset of seven tests, hydraulic conductivity ranged from 4.7×10^{-8} to 7.8×10^{-7} cm/sec at the higher stress levels, with a mean value of 2.9×10^{-7} cm/sec. The range for the lower stress level was 6.6×10^{-8} to 1.2×10^{-6} cm/sec, with a mean value of 5.6×10^{-7} cm/sec. Results are summarized in Table 2-8.

This data indicates that the Gilson Road QC results are likely to be conservative relative to actual in-situ conditions due to the lower than in-situ stresses used for QC testing. The data indicates a reduction in hydraulic conductivity by a factor of two. The actual reduction is probably even greater than this since the low stress data was obtained from samples which had previously been consolidated to higher in-situ stresses. Although the stresses were lowered to "top of wall" levels prior to testing, the soil fabric should still have reflected the previously higher stresses (i.e., the sample was overconsolidated). It would therefore yield hydraulic conductivities lower than those associated with identical samples which had not previously experienced the higher stress level.

Thus, the change in hydraulic conductivity with stress is likely to be greater than the above data indicates.

Five of the eleven "undisturbed" triaxial samples were then remixed and retested in the triaxial apparatus to assess whether the higher hydraulic conductivities measured in the "undisturbed" samples were related to the lesser degree of mixing when compared to the construction samples. Each sample was mixed thoroughly and retested in a "disturbed" state, modeling the triaxial tests performed during construction. The tests were run at low confining stresses. Test results, summarized in Table 2-8, indicated that the additional mixing lead to lower hydraulic conductivities in all five cases and by approximately one order of magnitude in three of the cases. Furthermore, the results on the remixed samples were consistent with the results from the samples obtained during construction, ranging from 2.6 x 10-8 cm/sec to 4.0 x 10^{-7} cm/sec.

API fixed-ring tests were subsequently performed on each of the eleven samples after remixing. These tests also yielded values consistent with those obtained during construction. Results ranged from 2.8 x 10^{-8} cm/sec to 2.8 x 10^{-7} cm/sec with a mean value of 1.0 x 10^{-7} cm/sec (refer to Table 2-8). Once again, remixing resulted in significantly lower hydraulic conductivities as compared to the undisturbed triaxial tests. Furthermore, remixed API fixed-ring tests yielded results very compatible to remixed, low stress triaxial tests (within an average factor of two). The additional mixing utilized to prepare the API fixed-ring samples from the remixed triaxial samples resulted in a further reduction in hydraulic conductivity.

Therefore, the results from the triaxial tests and API fixed-ring tests on the remixed post-construction samples indicate that remixing beyond the level achieved in the field to obtain duplicate sample splits may result in a lower hydraulic conductivity than that which exists in the field. This reduction in hydraulic conductivity for the Gilson Road site was nearly one-half an order of magnitude as based on the five backfill samples which were tested in the triaxial apparatus, first "undisturbed" and then remixed. (The average hydraulic conductivity of the "undisturbed" samples was 6.5 x 10^{-7} cm/sec and the remixed samples was 1.5 x 10^{-7} cm/sec.)

2.4.1.3 Evaluation of the Validity of the QC Construction Data

The primary purpose of the data obtained from post-construction samples is to validate the use of QC construction data for evaluating wall performance. Although the API fixed-ring test has been established as a reliable and accurate quality control procedure (Section 2.1), the validity of the QC construction data at the Gilson Road site also depends upon a viable comparison of test procedures and conditions with actual field conditions (i.e., in-situ stresses, no remixing), which are more closely reflected in the post-construction testing. Based on the post-construction data, two quality control testing procedural details have been identified which may result in the hydraulic conductivity of the Gilson Road QC construction samples being unrepresentative of that actually attained in the field.

The first such detail is the additional mixing of the backfill after sampling to increase homogeneity prior to splitting the sample for duplicate testing. The post-construction data indicate that this may have resulted in nearly a one-half order of magnitude, unconservative error in the Gilson Road QC construction data. This error can be easily avoided in future projects by eliminating the remixing step and accepting some variability in the duplicate sample aliquots. The second procedural step involves testing the samples at "top of wall" stresses in recognition of the indeterminancy of final backfill vertical location during sampling. The above data indicate that this procedure resulted in greater than a one-quarter order of magnitude conservative error in the Gilson Road QC data. This conservative bias is an unavoidable but desirable facet of the QC testing program.

The combination of the above two factors indicates that the Gilson Road construction QC data is likely unconservative by a factor of approximately two. Again, it is emphasized the elimination of the remixing step on future projects should result in a somewhat conservative bias to the QC data.

2.4.1.4 Implication of QC Data on Containment Performance

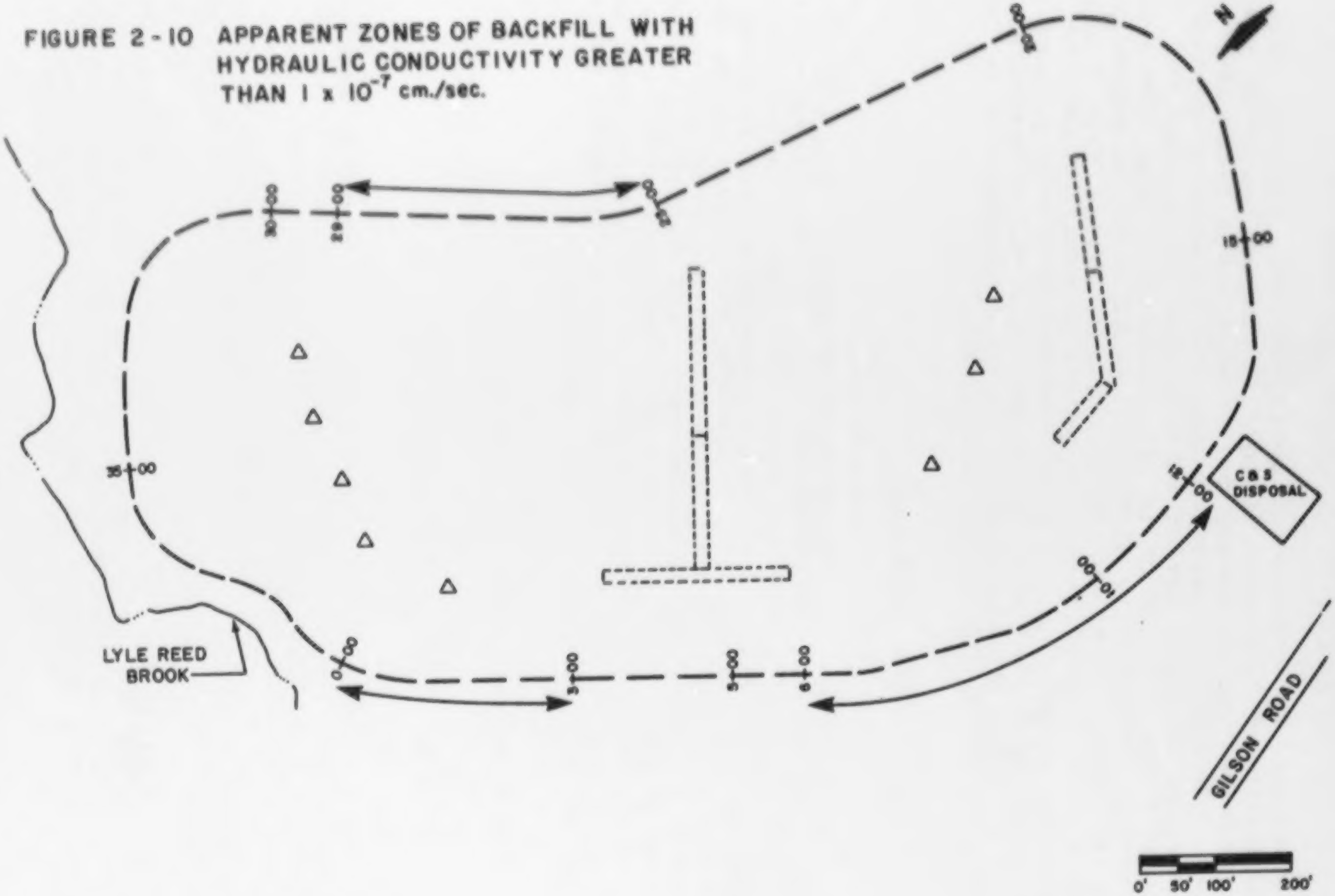
The quality control data obtained during construction indicated at the time that the hydraulic conductivity specification was generally satisfied while the percent bentonite specification was not. Although the hydraulic conductivity data indicated that approximately 22 percent of the samples tested had a hydraulic conductivity in excess of the maximum specified value of 1.0 x 10^{-7} cm/sec, the higher values were judged to have little impact on wall performance. The most important considerations in assessing the impact of the higher than specified hydraulic conductivities were the magnitude of the variation from the specified limit and the location along the wall alignment of the higher hydraulic conductivity zones. The average hydraulic conductivity of the nineteen tests above 1.0 x 10^{-7}cm/sec was 2.1 x 10^{-7} cm/sec. This margin is small relative to the four to five order of magnitude difference between backfill hydraulic conductivity and the hydraulic conductivity of the natural soil deposits (2 x 10^{-7} cm/sec vs. 1 x 10^{-3} to 1 x 10^{-2} cm/sec). It should also be recognized that the samples were tested at conservatively low confining stresses, corresponding to "top of wall" conditions. It is likely that some of these samples reflected backfill which ultimately advanced to a deeper location in the wall and was subjected to higher stresses in-situ than those in the test. Since an increase in confining stress would cause a reduction in hydraulic conductivity, the in-situ hydraulic conductivity is likely lower than the tested values.

The major impact of the higher permeability zones is that groundwater flow into the containment area, induced by pumping, will be marginally greater in these areas. Overall, however, as the average wall hydraulic conductivity was 5 x 10^{-8} cm/sec, groundwater inflow through the wall would be less than anticipated for a wall with an average hydraulic conductivity of 1 x 10^{-7} cm/sec. The presence of the bentonite filter cake along trench walls, not previously mentioned should also help compensate for marginally high hydraulic conductivity zones in the backfill.

In addition to the relatively small degree by which the backfill exceeded the specification limit, the higher hydraulic conductivity zones appear to be located in areas of relatively low hydrologic impact. As indicated, these areas include from station 0+00 to 3+00, 6+00 to 12+00, and 25+00 to 29+00 (see Figure 2-10). The direction of groundwater flow is predominantly parallel to these sections of the wall, resulting in comparatively low gradients across the wall. (Areas of largest gradients would be those sections perpendicular or nearly so to flow, approximately from station 13+00 to 19+00 and from station 32+00 to 0+00). The low gradients would result in comparatively low flow quantities across the wall.

The subsequent quality control testing program on samples obtained from the completed wall indicated that the construction

FIGURE 2-10 APPARENT ZONES OF BACKFILL WITH HYDRAULIC CONDUCTIVITY GREATER THAN 1 x 10^{-7} cm./sec.


N
20+00
15+00
12+00
10+00
6+00
5+00
3+00
0+00
35+00
30+00
29+00
25+00
C&S DISPOSAL
GILSON ROAD
LYLE REED BROOK
0' 50' 100' 200'

QC data may be somewhat unconservative. Average hydraulic conductivity of "undisturbed" backfill samples tested at in-situ stresses was approximately two times greater than that of the same sample remixed, suggesting that the hydraulic conductivities of samples obtained during construction may be half the actual hydraulic conductivity of in-situ backfill. Thus, the average hydraulic conductivity of the wall backfill may be approximately 1×10^{-7} cm/sec, the specified maximum, rather than 5×10^{-8} cm/sec as indicated by the construction data. In addition, the percentage of data above 1×10^{-7} cm/sec is likely greater than the 22 percent indicated during construction. Nevertheless, the somewhat higher average backfill hydraulic conductivity is judged to have little impact on wall performance.

2.5 Quality Control Guidelines

The previous sections have focused on the use of API fixed-ring testing and methylene blue titration for quality control of soil/bentonite backfill. These tests, however, represent only part of the quality control effort required for cutoff wall construction. In addition to further testing of the backfill, the quality control program must also address the support slurry, trench excavation, and backfill placement, because all of these aspects will affect the quality of the finished product. Specific guidelines for each of these areas are presented in the sections following a discussion of broad guidelines to quality control testing in general.

2.5.1 General Quality Control Guidelines

2.5.1.1 Test Requirements

The quality control program should be based on measurable indices of wall construction for which minimum design standards have been established in the project specifications. The contractor's workmanship can then be evaluated on the basis of conformance to these standards. Field tests used to measure these indices must meet four criteria to be acceptable for field quality control:

1. Timeliness: QC testing must be "real time" in nature to allow the timely modification of contractor's procedures early in the construction process, before inadequacies become pervasive and jeopardize the effectiveness of the wall. Tests which require several days to yield a

result do not allow early detection of a potential problem.

2. Accuracy/Precision: The tests specified must be sufficiently accurate and precise (repeatable) to yield a meaningful representation of the measured parameter or property.

3. Simplicity: QC testing must be based on simple equipment and procedures to allow execution in the field.

4. Low Cost: Testing must be inexpensive enough to allow a statistically significant number of tests to be performed.

As discussed in previous sections, the API fixed-ring hydraulic conductivity test and methylene blue tests both meet these criteria. Other quality control tests discussed in the following sections meet these criteria as well.

2.5.1.2 Quality Control Execution

In the past, field control has often been the sole responsibility of the contractor. This was a reflection, in part, of performance-based specifications which indiscriminately assign all responsibility for satisfactory cutoff construction to the contractor. However, because the contractor's primary interest is in getting the job done, QC testing is often relegated to a secondary importance. With limited construction staffing when a construction schedule slips, the priority of the contractor shifts to increasing production rather than quality.

QC testing performed by the contractor tends to serve as a check on work completed rather than a control over work to be done, analogous to the testing of concrete cylinders during building construction. Therefore, tests are often performed after a substantial portion of work is done, too late to have a significant impact on construction. Because the objective of quality control testing is not so much to document construction as to identify deficiencies in construction procedures as early as possible, "check" testing is an insufficient means of quality control. For quality control testing to effectively serve as a control over construction procedures, the tests must be performed and reviewed on a timely basis, ideally by a qualified representative of the owner (the design engineer) rather than the contractor. The engineer as quality control agent provides a number of important advantages as compared to the contractor:

A. Understanding of Relationship between Design and QC Data:

The design engineer has the highest degree of understanding of the assumptions and constraints under which the project was designed. As such, he is the most capable of assessing the meaning of the QC testing data. In addition, unanticipated variability in subsurface conditions, etc., which can only be determined via construction observation, may in themselves appear inconsequential (to the contractor) but may have a major impact on overall performance if not accounted for in design. Again, the design engineer is best equipped to evaluate this information as it affects the specific construction at hand and/or subsequent construction phases. The contractor may not have full knowledge of all the design considerations and may not as readily appreciate the significance of seemingly aberrant test results.

B. Modifications:

Modification of QC procedures is relatively easily accomplished by the engineer in response to unanticipated contingencies. On the other hand, a contractor following QC procedures outlined in the specifications may not be in a position to judge what modifications are necessary if unexpected contingencies arise.

C. Responsiveness:

Inasmuch as a QC team of engineers is not burdened with responsibility for the actual construction, their efforts can be concentrated on taking samples, performing tests, and providing real time data. If difficulties develop during construction, the contractor may try to reallocate QC testing manpower to activities directly related to "getting the job done." Hence, during precisely the period when QC testing may be most important, its emphasis may be reduced.

D. Objectivity:

If the contractor is responsible for QC testing, objectivity during sample selection and test execution may not exist to a sufficient degree.

E. Quality:

If the contractor is responsible for QC testing, obtaining quality testing equipment and adequately experienced technicians are not guaranteed.

The contractor should do limited testing of his own, but it should not be incorporated in the engineer's quality control program. The contractor's data should be gathered for his own use should a dispute arise with the engineer's quality control data.

2.5.2 QC Guidelines for Support Slurry

During cutoff wall construction, the stability of the trench excavation depends to a large part on maintaining a positive head of a bentonite slurry within the trench.

Design parameters for the slurry include:

°	Unit Weight	- the slurry density is an indirect indicator of bentonite content; the density should be slightly higher than that of water, while not being so heavy as to impede the movement of the advancing backfill face.
°	Viscosity	- the viscosity of the slurry is an indicator of the rate at which the filter cake will develop along trench walls; the higher the viscosity the faster the filter cake will form due to higher bentonite concentration in the slurry.
°	Filtrate Loss	- filtrate loss is also an indicator of the capacity of the slurry to form a filter cake of bentonite particles along the trench walls.

Because the support slurry is ultimately displaced and not a permanent element of the completed cutoff, its impact on wall performance is significantly less than that of, say, backfill preparation or placement. The filter cake which remains does contribute to overall wall impermeability, typically having a hydraulic conductivity of 1×10^{-8} cm/sec or less. However, the contribution of the filter cake is not accounted for when evaluating wall performance and is thus regarded as a safety factor. The primary impact of an inadequate slurry is trench collapse, which poses construction problems (i.e., delays), but does not necessarily have a long-term effect if the collapsed section is reexcavated properly. Thus, the contractor will be more concerned with a trench collapse, or the potential for one, than the engineer.

Accordingly, quality control of the support slurry is often the contractor's highest priority during construction. The frequency of his slurry testing typically far exceeds that of other quality control tests because the consequence of inadequate slurry is costly in terms of time lost for reexcavation. At the same time, because of the contractor's concern with construction delays and the comparatively low impact of slurry quality on the completed wall, the engineer need not place great emphasis on slurry testing when planning his quality control program.

The engineer should include within his quality control program a series of tests on slurry, but the required testing frequency can usually be less than that adopted by the contractor. Slurry quality control tests include: unit weight, viscosity, and filtrate loss. Their use is described in the following paragraphs.

2.5.2.1 Unit Weight

Unit weight of the slurry may be determined by numerous means of weight/volume measurements. However, unit weight is commonly determined using the mud balance test, which consists of a specially designed scale calibrated to indicate density when loaded with a prescribed volume of slurry. As indicated, the slurry should always have a density greater than that of water while not being so dense as to impede backfill placement.

Execution of the test is straightforward, and the verification of a minimum unit weight relatively simple. A slurry sample obtained from the top of the trench is sufficient to establish whether the slurry density is higher than 62.4 pcf. Slurry unit weight generally increases with depth, however, due to the slow sedimentation of entrained sand and, in certain cases, the settling of flocculated bentonite particles. Determination of maximum slurry unit weight must then be based on a sample obtained from the bottom of the trench. Collecting such a sample can be difficult.

The use of a backhoe bucket to obtain a sample from the bottom of the trench is generally inadequate. By the time the bucket emerges from the trench bottom, the slurry in the bucket is most representative of slurry from the top of trench, particularly if the bucket is designed with holes to drain water, as is common. Therefore, alternative means of sample collection must be used. A sampling device which traps slurry from the trench bottom is necessary. Groundwater samplers with ballcheck valves are commercially available, but are typically inadequate for this use. A slurry sample which contains entrained sand particles may interfere with proper seating of the valve. In addition, if the slurry is heavy and highly viscous, it will not pass the sampler orifice and the sampler will displace the heavy slurry.

2.5.2.2 Viscosity

Viscosity is determined using the Marsh Funnel test which measures the time for a fixed volume of slurry to empty from a standard Marsh Funnel. Typically, a minimum time of forty seconds is specified to achieve the proper viscosity. A contractor will try to maintain a Marsh Funnel time as close to forty seconds as possible without dropping below forty seconds. Because viscosity is proportional to the bentonite content and barrel yield[2] of the slurry, a viscosity significantly in excess of the specified value is indicative of excess bentonite. Because the cost of bentonite generally represents the highest materials cost, the contractor will seek to use the minimum bentonite required.

2.5.2.3 Filtrate Loss

Filtrate loss tests are run in the API filter press cell. The standard criteria for support slurry are less than 30 cc of filtrate loss when subjected to a 100 psi pressure for thirty minutes. The tests are typically run on fresh mixed slurry prior to the introduction into the trench. A test on a sample of the same slurry taken from the trench would yield a lower volume of filtrate due to filtrate loss occurring in the trench. It should also be added, however, that the filtrate loss test is a good indicator of adverse chemical effects on the slurry. Groundwater contaminants resulting in bentonite flocculation would lead to a slurry with a relatively high value for filtrate loss.

---

[2] Barrel yield is defined as the number of 55-gallon drums of bentonite slurry produced by one ton of dry bentonite.

2.5.3 QC Guidelines for Trench Excavation

Control aspects of slurry trench excavation include:

- trench verticality
- trench continuity
- depth of penetration into key (aquiclude)
- bottom cleaning
- trench stability

Quality control of trench excavation generally receives little emphasis, particularly when compared with quality control testing of slurry and backfill. To some degree, this is because the excavation process is regarded as a relatively common construction operation. This attitude also reflects the fact that slurry trench excavation is performed "blindly:" there is no way to observe the sides or bottom of the trench. Another reason is that quality control of the trench excavation is largely subjective. Aside from trench width and depth, there are no distinct, measurable parameters.

As compared with quality control of slurry or backfill, quality control of excavation consists more of a series of measurements than tests. Methods of evaluating trench verticality, continuity, depth of penetration into key, the condition of the bottom, and trench stability are discussed in the following paragraphs.

2.5.3.1 Verticality

Verticality of the trench is assessed by measuring the verticality of the excavation equipment. The measurement is best made using a 4-foot level on the backhoe boom or on the backhoe tracks. (The backhoe typically sits on a compacted clay working pad constructed along the trench alignment. The pad is primarily intended to provide stable walls at the top of the trench).

Verticality is not generally regarded as a critical aspect of wall construction, particularly to the contractor, since the wall does not have to be perfectly vertical to function as designed. However, wall verticality is essential to post-construction investigation, particularly if backfill samples are required from the completed wall and the wall is deep. A small deviation of several degrees from the vertical at ground surface can result in deviation of several feet at the bottom of a 50-foot trench. The time and money spent in "finding" the wall at depth via borings can be a major obstacle to post-construction verification.

Therefore, a horizontal working pad should be maintained during construction. If the excavating equipment deviates by more than 1 degree from vertical, the pad should be regroomed to restore levelness. If the pad is properly established, however, there should be little need to regroom because there should be little construction activity along the wall alignment prior to excavation.

2.5.3.2 Continuity

Continuity of the trench is largely verified by the sweep of the backhoe stick. Especially along relatively straight sections of wall, the action of the backhoe should indicate the continuity of the excavation. Problems are most likely to arise at sharp bends in the wall alignment. The continuity of the trench may be affected by a discontinuous, or interrupted,excavation procedure. The contractor may have to reposition his backhoe several times to negotiate the bend. Each repositioning creates a potential for a discontinuity.

To reduce the risk of discontinuity, the contractor can overexcavate one leg of the trench and then start a new trench angled anywhere from 45 to 90 degrees from the old. The new trench should be started sufficiently behind the old trench to intersect at full depth. The main disadvantage associated with this type of overexcavation is that backfill placement is more difficult to accomplish properly.

The quality control engineer can monitor trench continuity by dragging a rigid, vertical probe along the trench bottom. A break in trench continuity will block the advance of the probe.

2.5.3.3 Key Penetration

Adequate penetration of the trench into a naturally occurring aquiclude is fundamental to the performance of a cutoff wall. Thus, it is critical to know at what depth the aquiclude is first encountered in order to determine the minimum acceptable depth of trench to satisfy required penetration. The contractor typically relies on his backhoe operator to determine when the aquiclude is encountered and when sufficient penetration is achieved. The depth of the trench is then commonly checked by the contractor using a weighted rope calibrated in foot increments.

The quality control engineer must employ a more precise method of measurement. The measurement tool should be a rigid probe calibrated in foot increments. When the backhoe operator indicates he has encountered the aquiclude, a depth measurement should be taken. Verification that the aquiclude has been reached is a simple matter of checking the trench spoil. The aquiclude, when probed, should also "feel" different than the generally coarser soils which overly it, particularly if the aquiclude is a clay or silt. This first measurement should become the record depth to top of aquiclude even if the backhoe may have penetrated 6 to 12 inches into it. Key penetration should be referenced to this depth. As the contractor continues to excavate into the aquiclude, subsequent measurements should be taken until the minimum specified depth of penetration is achieved. The depth measurements should be referenced to pre-established stations along the wall alignment to maintain a record of excavation depth and key penetration depth.

2.5.3.4 Key Cleaning

The removal of sediment from the key zone prior to backfill placement is important for two reasons. First, the key penetration is effectively reduced if a portion of the key zone contains coarse sands which have sedimented out of the slurry. The reduced penetration may jeopardize the integrity of the cutoff. Second, sediment at the trench bottom can be pushed as a wave by the toe of the advancing backfill slope to the extent that the sediment mounds and becomes incorporated in the backfill. The advancing backfill pushes this pile of sediment until it becomes too heavy to move. The backfill then rides over the mound of sediment, leaving a "window" within the backfill. This is most likely to occur at a bend in the wall alignment.

The removal of sediment from the trench bottom is most-commonly accomplished by the backhoe bucket. The quality control engineer, however, must have the ability to "feel" the trench bottom himself to assess the effectiveness of the cleaning. This evaluation can generally be made using a rigid probe. Also, the variation in the vibration of the probe as it passes through the slurry and sediment can be detected. It is unlikely that this determination can be made by the contractor using the backhoe bucket because the heavy equipment is not sensitive enough to detect the transition from slurry to sediment.

The probe technique can also be used to detect the build-up of sediment at the toe of the advancing backfill slope. Frequent probing for this problem should be incorporated into quality control procedures. As required, the contractor should excavate the mound of sediment before it becomes trapped within the backfill.

2.5.3.5 Stability

Stability of the uppermost section of the trench is relatively easy to monitor. Typically, a failure of the excavation will be signaled by the development of tension cracks, roughly parallel to the trench alignment. The failure itself will consist of a conchoidal wedge of soil which slides into the trench. The remedy in such a case is simply to excavate the fallen soil.

Stability of the trench at greater depths, when there may be no signs of failure at ground surface, is more difficult to assess. The sloughing of a trench wall can occur below the slurry level and may involve no visible movement of soil at the surface. If the sloughing occurs in a section of the trench that has not been completely excavated or cleaned, the sloughed material will most likely be excavated. If the sloughing occurs in a section of trench through which the backfill is advancing, however, the sloughed soil will get trapped within the backfill, leaving a high hydraulic conductivity window.

Though there are no direct means of assessing trench stability below the slurry level, the quality control engineer can detect sloughing failures using an indirect approach. This consists of frequently monitoring the profile of the backfill slope. The backfill should normally slump forward in the trench on a relatively consistent slope. This can be verified by probing at intervals along the trench alignment. The frequency of probing and spacing will be determined on a case-by-case basis. A sloughing failure would deposit soil on the backfill slope and alter its profile. The mound of soil created by the failure should be detectable by probing, showing up as an irregularity in the backfill profile. Profiling at the beginning and end of each working day will allow the timely detection of a sloughing failure.

2.5.4 QC Guidelines for Backfill

The focal point of the engineer's quality control program should be the backfill. Further, QC of backfill should involve a series of tests which address aspects of both backfill placement and long-term performance.

QC tests related to backfill placement include:

- Unit Weight - the backfill must be sufficiently dense to displace the support slurry.
- Slump - the backfill must have a high enough slump to "flow" into the trench without being too fluid and thus mixing with the support slurry.

QC tests related to long-term backfill performance include:

- Hydraulic Conductivity - API fixed-ring testing is the most direct quality control measure to assess wall performance.
- Methylene Blue Titration - bentonite content determination should be used to supplement API fixed-ring results particularly if hydraulic conductivity is too high.
- Gradation - gradation testing during construction is perhaps the least critical quality control test since backfill components and proportioning should have already been determined during design testing; gradation testing does, however, provide an ongoing check on fines content.

2.5.4.1 Unit Weight

Determining the unit weight of the backfill involves a straightforward weight-volume calculation. Most critical is obtaining a large enough sample to yield a representative unit weight. Thus, the size of the sample must be large relative to the largest particle size in the backfill. A standard concrete cylinder should generally suffice.

Unit weight testing rarely has an impact on construction procedures. Most backfill mixes, whether granular or clay-based, will easily satisfy the minimum unit weight requirement with no special effort.

2.5.4.2 Slump

Slump testing of backfill is performed using the same procedure as slump testing of concrete (ASTM C-143-78). The minimum and maximum slump values should be selected based on backfill placement considerations. A low slump mix will not "flow" easily into the trench and will result in relatively steep backfill slopes within the trench. The steeper the slope, the greater the probability of a slope failure which could potentially trap a pocket of slurry. A high slump mix will move easily into the trench and result in a relatively flat backfill slope. The greatest risk of a high slump mix is that the slurry will not be positively displaced and there may be some mixing of the slurry and backfill. A secondary risk is that a relatively flat backfill slope will result and thus long sections of trench will be open for an inordinately long time, increasing the potential for a sloughing failure. The deeper the trench, the more likely the chance of a failure.

Assuming that the contractor has prepared the backfill using the proper proportions of backfill components, results of the slump test should be used as the main determinant of whether the backfill is ready for placement in the trench. It is not practical to wait (even several hours) for the results of hydraulic conductivity results on the backfill. Therefore, once the backfill has achieved an acceptable slump and visual homogeneity, placement is allowed.

2.5.4.3 API Fixed-Ring Hydraulic Conductivity Testing

Guidelines for use of the API fixed-ring test for quality control were outlined in Section 2.1.3. To summarize:

- At the design stage and during the early stages of construction the site specific accuracy of the API fixed-ring test should be assessed by comparing APA fixed-ring test results to triaxial hydraulic conductivity results on identical backfill samples.
- During construction, samples obtained for API fixed-ring testing should be transported and placed in the test apparatus with as little disturbance as possible. Specifically, no additional mixing should take place.

2.5.4.4 Methylene Blue Titration for Bentonite Content

Guidelines for use of the methylene blue test were outlined in Section 2.2.3. Besides its obvious value of determining conformance with specifications, the methylene blue test should be used to complement API fixed-ring testing. If backfill hydraulic conductivity is too high, it is necessary to identify the reason. Because insufficient bentonite content is the most probable cause of a high hydraulic conductivity, methylene blue testing can be used to verify bentonite content.

2.5.4.5 Gradation

Gradation testing of backfill samples involves conventional soil testing procedures. Because gradation specifications for backfill are of necessity broad, the primary value of gradation testing is in the determination of percent fines. As with methylene blue testing, gradation testing should be considered a complement to API fixed-ring hydraulic conductivity testing, particularly where API fixed-ring tests yield unacceptable results.

2.6 Conclusions

The first phase of this project focused on the evaluation of two quality control procedures for soil/bentonite cutoff walls: API fixed-ring hydraulic conductivity testing and methylene blue titration for determination of bentonite content. These procedures were adapted for use at the Gilson Road site in 1982. As part of the project, testing procedures were refined and standardized for future use. Following assessment of these procedures, the quality control data obtained at Gilson Road was reviewed to assess conformance to specifications. Based on experiences gained at the Gilson Road site and other subsequent cutoff wall projects, guidelines for additional aspects of quality control were developed.

Major conclusions of this phase of the research effort are:

- Both the API fixed-ring hydraulic conductivity test and methylene blue test are suitable for field quality control of cutoff wall construction; they are sufficiently accurate and precise to allow a meaningful evaluation of future wall performance, they require relatively short testing times, they are relatively simple to perform and they are relatively inexpensive on a per-test basis.

- A quantitative correlation between hydraulic conductivity and backfill gradation characteristics did not emerge from a statistical evaluation of the data obtained at the Gilson Road site; the importance of such a correlation, however, diminished over the course of the research effort as the suitability of API fixed-ring testing and methylene blue testing was confirmed.
- Hydraulic conductivity data obtained at the Gilson Road site indicate that project specifications for the completed wall were generally satisfied; approximately 20 percent of the data were in excess of the maximum allowable limit, but by a margin deemed to have minor impact on overall wall performance; the bentonite contents for these samples were generally below the minimum specification limit, but were generally above the required minimum based on a design testing.

SECTION 3

WINDOW DETECTION VIA PIEZOCONE SOUNDING

Appropriate use of quality control field testing allows a high degree of certainty to be developed with respect to the adequacy of soil/bentonite backfill proportioning and mixing during cutoff wall construction (Section 2). This, coupled with data obtained during the backfill design phase prior to construction (Schulze, 1984), provides confidence that the intact backfill will meet the long term hydraulic conductivity specifications once it is placed in the trench. However, placement of the backfill in the trench results in a set of inherent conditions which can allow entrapment of non-backfill material ("windows") in the cutoff wall. While the intact backfill may meet specification, the windows are typically of a higher hydraulic conductivity and thus degrade the overall performance of the containment.

The formation of windows may result from various scenarios such as:

- Sloughing of in-situ soils from the sides of the trench onto the advancing backfill face.

- Sedimentation of relatively course material, temporarily suspended in the support slurry during excavation, onto the backfill face.

- Inclusion of pockets of bentonite support slurry during slurry displacement by the backfill and

- Failure of the excavation tools to remove all the in-situ material and/or trench sediment from the key profile, particularly when keying to the top of bedrock.

These occurrences can be due to either geologic conditions, such as highly pervious open worked in-situ soils causing local instabilities and thus sloughing, or construction inadequacies such as insufficient or excessive backfill fluidity.

Based on the probable mechanisms leading to window formation, they are expected to be relatively small in size;

hypothesized order of magnitude less than 10 to 100 square feet in profile dimension. Despite their small size, windows can have a significant impact on the overall efficiency of the cutoff wall barrier. This is a direct result of the groundwater mounding and thus increased gradient in the vicinity of the barrier[1] (Powers 1981).

The ability to cost-effectively detect such defects is instrumental to the verification of successful adherence to the performance specification for backfill homogeneity should a post-construction verification program become necessary. Furthermore, the demonstration of a capability for post-construction window detection may provide added incentive to the contractor to strive for the highest quality of workmanship during construction.

The most direct method of window detection is to obtain undisturbed samples of the completed cutoff wall. A quantitative assessment of the hydraulic conductivity of the window material can then be obtained via laboratory testing. However, due to the anticipated small size of a typical window as compared to the cross sectional area of the cutoff wall,[2] a large number of samples are required to detect a statistically significant percentage of the windows. The high unit cost of sample retrieval and laboratory testing therefore render this solution impractical[3].

As an alternative to direct sampling, the feasibility of remote sensing geophysical techniques was considered. Down-hole and cross-hole techniques have gained wide acceptance by the oil related industries. More recently, surface techniques have been developed for hazardous waste investigative work. However, much of the current technology was judged to be unacceptable for detection of anomalies in soil/bentonite cutoff walls as based on theoretical considerations. Limitations of these techniques are discussed in Appendix C.

---

[1] The mounding and increased gradient caused by the intact portions of the barrier elevate the flux through the windows above that which would be calculated based on the cross-section of the window and the gradient which existed before barrier construction.

[2] For the Gilson Road project, a 100 square foot window would account for less than five one-hundredths of one percent of the cross sectional area of the 4,000-foot long wall.

[3] Undisturbed shelby tube samples taken at 10-foot depth intervals at 50-foot centers along the Gilson Road cutoff wall would cost approximately $400,000 including triaxial testing. This figure approaches one-half the cost of wall construction.

The considerable experience existing in the geotechnical exploration and geophysical testing fields indicated that piezocone equipment might be ideally suited for window detection. Commercially available instruments have been routinely used for over fifteen years for foundation design and off-shore work in soft soils. Originally developed for stratigraphic identification, these instruments provide continuous soil index data as they are advanced from the ground surface. Their rapid penetration rate also offer the potential for cost-efficient continuous soundings at many locations along a cutoff wall. The primary objective of Phase Two was therefore, to evaluate the suitability of electronic piezocone instrumentation for window detection.

The first step in this phase was to evaluate the need for modification of existing equipment to make it more suitable for detection of windows in soil/bentonite cutoff walls (Sections 3.1 and 3.2). The second step was to construct a cutoff wall test section incorporating various types of windows at known locations (Section 3.3). The ability of the piezocone to detect windows in the test section was then evaluated as the third step in the research (Section 3.3). Steps four and five encompassed use of the piezocone in the actual Gilson Road containment wall (Section 3.4) and obtaining undisturbed samples of materials identified as potential windows (Section 3.5).

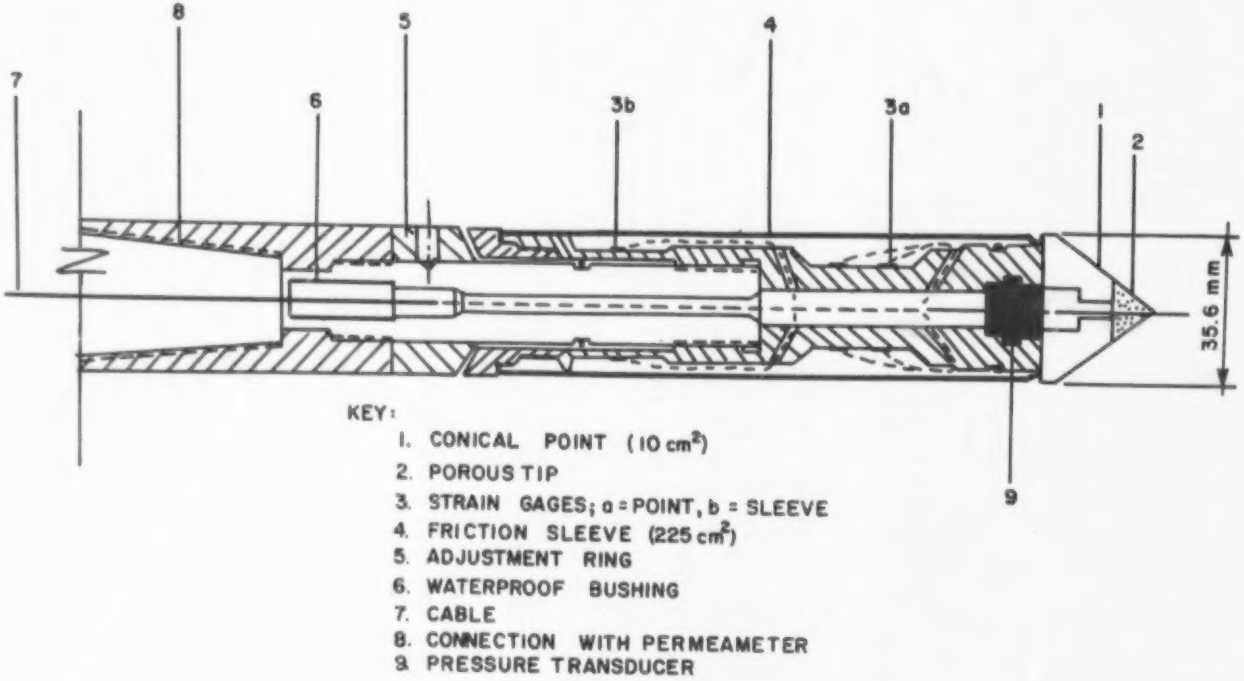
3.1 Piezocone Operating Principles

3.1.1 Standard Instrument Configuration

Current electronic piezocone instrumentation is an outgrowth of the mechanical Dutch Cone equipment used in Europe for many years (Sanglerat 1972) and the piezoprobe developed in the early 1970's (Wissa 1975). The piezocone used for this project was developed by Geotechniques International, Inc. and is depicted in Figure 3-1. The instrument has a 60 degree apex angle conical point, 3.56 cm in diameter, with a projected area of 10 cm^2 as based on the Fugro design. Overall probe length is approximately 50 cm.

The conical point of the instrument is connected to a load cell which allows measurement of the point resistance developed during sounding. The end of the point houses a porous section

FIG. 3-1 SCHEMATIC OF PIEZOCONE INSTRUMENT


7
8
6
5
3b
4
3a
1
2
35.6 mm
9
KEY:
1. CONICAL POINT (10 cm^2)
2. POROUS TIP
3. STRAIN GAGES; a = POINT, b = SLEEVE
4. FRICTION SLEEVE (225 cm^2)
5. ADJUSTMENT RING
6. WATERPROOF BUSHING
7. CABLE
8. CONNECTION WITH PERMEAMETER
9. PRESSURE TRANSDUCER

which is hydraulically connected to a pressure transducer for measuring excess pore pressures generated during the sounding. The frictional and/or adhesion force on the side of the instrument is measured by an independent load cell. The load cell is connected to a 225 cm^2 area friction sleeve which is 20 cm long and is positioned immediately behind the point. The data generated by these transducers are transmitted to the data acquisition system at the surface via an electrical cable. Additional description of the standard piezocone, its development, and application can be found in Sanglerat 1972.

3.1.2 Instrument Modification

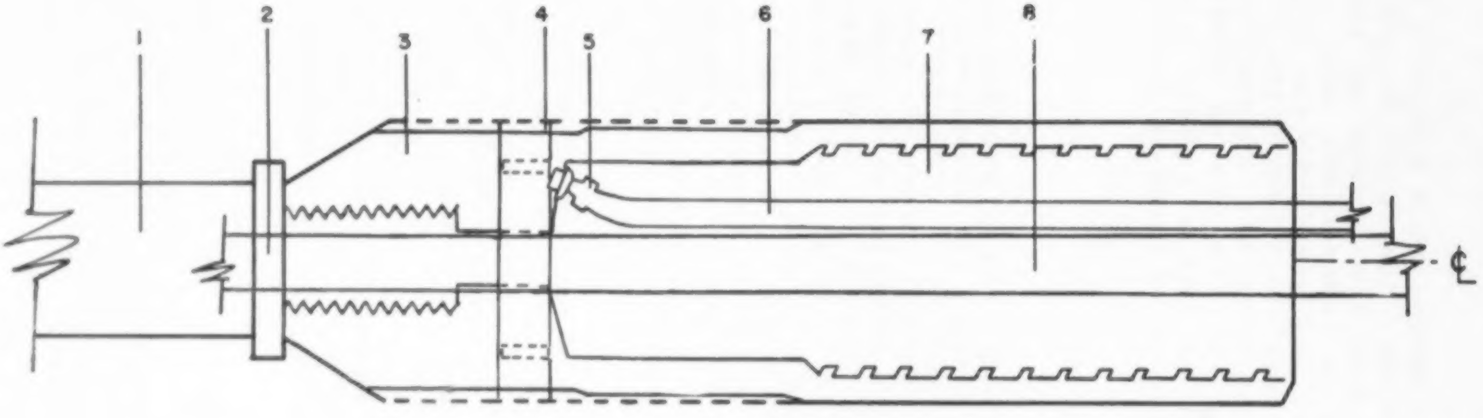
As part of this effort, the piezocone was modified to include an additional data channel for measurement of a "relative hydraulic conductivity index." The rationale for this modification was based on recognition that the backfill hydraulic conductivity was the primary parameter of interest. Furthermore, the standard piezocone data channels for point resistance, excess pore pressure, and side friction primarily respond to material density, consistency, and shear characteristics, rather than hydraulic conductivity. Therefore, a more direct indication of hydraulic conductivity was sought to custom tailor the piezocone to soil/bentonite backfill sounding. Inasmuch as a direct measure of in-situ backfill hydraulic conductivity was not possible, a relative quantitative index was considered appropriate.

The "permeameter" consisted of a 50 micron pore size ultra high molecular weight polyethylene porous stone. The stone was 23 cm^2 in exposed area and was located approximately 60 cm behind the point (see Figure 3-2). It was hydraulically isolated from the pore pressure transducer at the point of the piezocone and also from the annular space within the drill rods. The stone was maintained at a constant driving pressure[4] via a 1/4-inch diameter nylon tube which connected the stone to a pressurized water reservoir at the surface. The reservoir was pressurized via a precision regulator and a tank of compressed gas.

---

[4] The total driving pressure in the stone was increased as the depth of the instrument increased during a sounding. The total piezometric head in the stone was thus maintained at a constant preset level above the total piezometric head in the in-situ soils surrounding the cutoff wall as based on steady state hydrostatic conditions.

FIG. 3-2 SCHEMATIC OF PERMEAMETER



KEY:

1. PIEZOCONE
2. BOND BREAKER
3. PERMEAMETER
4. POROUS STONE
5. FITTING FOR WATER ENTRY TO STONE
6. 0.25 INCH NYLON TUBE
7. N W COUPLING TO DRILL STRING
8. PIEZOCONE CABLE

NOTE: DASHED LINE INDICATES INITIAL PROFILE.

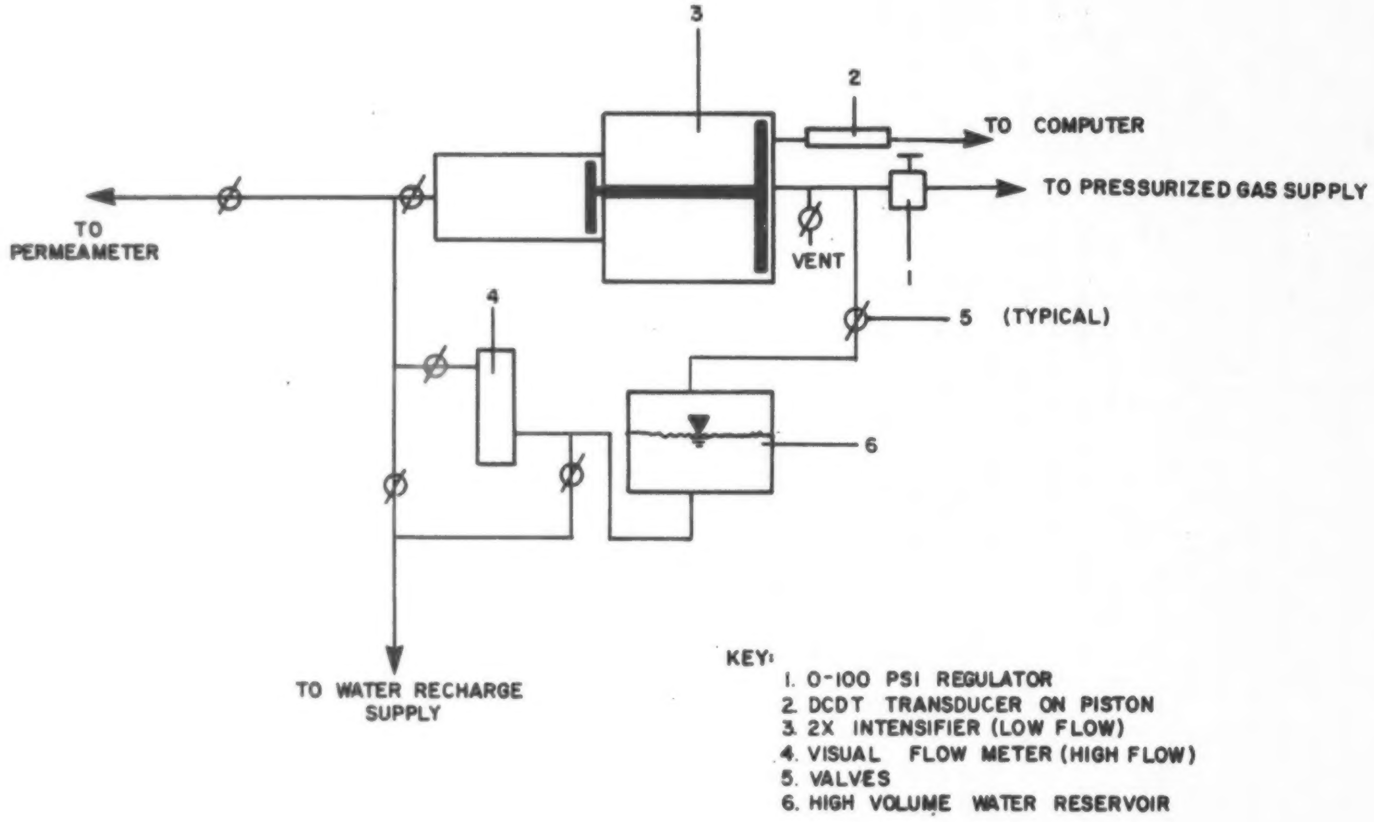
During a sounding, the volume of water which exited the stone into the soil/bentonite backfill was monitored versus depth of probe penetration. Flow volume monitoring for the low flow rates associated with the relatively impermeable backfill was accomplished using a commercially available unit which is commonly employed for triaxial testing (manufactured by Geotechniques International, Inc., see Figure 3-3). This unit records the volume of water which is expelled from the surface reservoir via a sliding piston. The piston travel, which is directly proportional to volume, was monitored with a linear direct current displacement transducer (DCDT). The output of the DCDT was fed into the data acquisition system. The rate of flow into the backfill at various depths provided an index indicating relative backfill hydraulic conductivity. An auxiliary pressure application/flow measuring system was also incorporated into the equipment to monitor higher flow rates but at a reduced accuracy. This portion of the equipment was not used due to limitations in the rate of flow which could be transmitted through the porous stone.

3.1.3 Sounding Procedure

To execute a sounding, the piezocone was pushed into the formation via a drill rig at a constant rate of penetration (1 to 2 cm/sec). The sounding was stopped at 5-foot intervals to add drill rod. The depth of the piezocone was monitored by an external transducer which is connected to the drill string. Data output from this transducer was fed to the data acquisition system. The drilling procedure utilized during this project was in accordance with ASTM 3441.

Inasmuch as the permeameter portion of the instrument was not standard, procedures for its operation are not covered by ASTM. The procedure used is therefore described as follows. Initially, a determination of the magnitude of driving pressure to be applied to the stone was made. Selection of this pressure represents a tradeoff between two mutually exclusive objectives; 1) a high pressure provides high volume flow into the backfill which increases measurement accuracy and 2) a low pressure reduces the probability of hydraulically fracturing the formation or creating boundary flow along the drill string, particularly at shallow depths. The pressure utilized during the project described herein was generally 2.5 psi for the test section soundings and and 15 psi for the containment wall soundings. This driving pressure was applied to the stone prior to each sounding to ensure that the porous stone, pressurization and volume measuring systems were operating correctly. The rate of

FIG. 3-3 PERMEAMETER PRESSURIZATION/FLOW MEASUREMENT SYSTEM


3
2
TO COMPUTER
TO PRESSURIZED GAS SUPPLY
TO PERMEAMETER
VENT
1
4
5 (TYPICAL)
6
TO WATER RECHARGE SUPPLY
KEY:
1. 0-100 PSI REGULATOR
2. DCDT TRANSDUCER ON PISTON
3. 2X INTENSIFIER (LOW FLOW)
4. VISUAL FLOW METER (HIGH FLOW)
5. VALVES
6. HIGH VOLUME WATER RESERVOIR

flow was recorded and compared to previous data during each sounding. The driving pressure was maintained at a constant value throughout the sounding. During each drill rod addition (5-foot intervals), flow to the stone was terminated and the surface water supply reservoir was refilled.

Data from the three internal piezocone transducers (point resistance, dynamic pore pressure and side friction) and the two external transducers (permeameter flow and instrument depth) were transmitted through electrical cables as independent 4 to 20 milliamp analogue signals. The signals were processed to digital format by an Analog Devices A to D converter. The digitized data was stored directly in memory and subsequently on floppy disk using an Analog Devices Mac Sym 150 Computer.

3.1.4 Data Manipulation and Output Format

The telemetry data for point resistance, excess pore pressure, local friction and permeameter volume flow was initially in chronological format. A data reduction program then transformed the data into a depth correlated format[5]. The program also subtracted the value of the steady state pore pressure at each point in the formation surrounding the wall (computed assuming hydrostatic conditions) from the total pore pressure measured at the point of the piezocone. Excess pore pressure, as generated by quasi-static penetration[6], was thus computed. In addition, the program manipulated the independent transducer outputs to develop composite parameters. These included friction ratio (side friction divided by point resistance) and A factor (excess pore pressure divided by point resistance). The input data used for these computations was depth correlated and, as such, represented transducer response for the material at a given depth.

---

[5] The individual piezocone sensors, located along the 60 cm length of the instrument, transmit data immediately as it is taken. Therefore, at any instant of time, each sensor is responding to backfill characteristics at a different depth. To be useful, the data from the individual sensors must be shifted in time such that the individual parameter values correspond to the same backfill depth.

[6] As the cone penetrates the material of interest, it generates an excess pore pressure (a pore pressure different from that which existed before the sounding) due to cavity expansion displacement and shearing processes. This excess pore pressure varies in magnitude and can be either positive or negative depending on the material characteristics.

The data acquisition system performed the above described data reduction on a real time basis. As such, the output was displayed directly on a CRT screen to allow the operator to monitor piezocone operation during the sounding. The following data was presented on the CRT as plotted against a depth axis:

- Point resistance in kg/cm^2
- Local friction in kg/cm^2
- Excess pore pressure in kg/cm^2
- Friction ratio in percent
- A-factor in percent
- Permeameter volume displacement in cm^3/ft

At the end of the sounding, a paper copy of these plots was generated.

Each time the sounding was stopped to add another length of drill rod (5-foot intervals), the excess pore pressure at the piezocone point was monitored with time[7]. The excess pore pressure equilibration behavior thus generated provided additional data with respect to material characteristics. These data were initially presented in real time on the CRT and then as hard copy. The sounding was stopped and these data collected at points other than drill rod addition when desired by the operator.

3.2 Material Identification

Electrical piezocone equipment is well suited for differentiating between various soil types and thus determining the geologic stratigraphy encountered during a sounding. Material identification is accomplished through the use of empirical correlations which relate transducer output to material properties. The following sections first discuss the expected response for each individual transducer when sounding various soil types. It is recognized that each such index, by itself, does not provide definitive soil identification. However, the discussion serves to demonstrate the characteristics and range of responses to be expected. The relative output of multiple transducers, producing indices such as friction ratio and A factor is then presented as related to empirical correlations which have been developed for soil type identification. Finally, the expected response of the piezocone to intact soil/bentonite backfill and various potential window types is discussed.

---

[7] The excess pore pressure is generated by piezocone penetration of the material. Therefore, once penetration is stopped, the excess pore pressure decays to the steady state value which existed prior to the sounding.

3.2.1 Point Resistance

During the development of the electrical piezocone, the first index measured was point resistance. This is defined as the load required to penetrate the point into the material of interest[8] divided by the projected area of the point (typically 10 cm^2). In addition to the standardization of point area, the point apex angle has also been standardized at 60°. Penetration is controlled to a constant, relatively slow rate (1 to 2 cm/sec) and is therefore considered quasi-static. As such, dynamic inertia effects are assumed to be zero.

The penetration resistance encountered by the point is primarily related to the density and shear strength of the material penetrated. The greater the density and/or shear strength, the greater the magnitude of point resistance. This is true for both cohesive (clayey) soils and cohesionless (granular, non-plastic) soils. As such, point resistance by itself cannot be used to distinguish soil type. A loose/dense sand can yield the same point resistance as a soft/stiff clay depending on the relative degree of the material density/consistency.

In soils with a significant percentage of gravel size constituents, erratic and artificially high readings may be obtained locally. This typically occurs when the point engages a piece of gravel, particularly one with a projected area greater than 10 cm^2, and pushes it through the soil matrix. This problem is alleviated to some extent by averaging algorithms contained in the data reduction program[9].

---

[8] The load generated by point penetration is measured by a transducer connected directly to, and positioned immediately behind, the point. In this way, forces generated by side friction/adhesion on the sides of the instrument and the drill string is not measured.

[9] With the system used during this project, the analog transducer output is digitized at 60 hertz. The individual digital values are then averaged over a one-quarter second time span (penetration depth of 1/4 to 1/2 cm) for data presentation.

3.2.2 Local Friction

The second parameter to be incorporated into cone penetrometer instruments was measurement of local friction. Local friction is defined as the vertical shear load engaged by the friction sleeve during penetration divided by the friction sleeve area (typically 150 cm^2, but 225 cm^2 for the instrument used during this project in recognition of the probable low shear values). The friction sleeve is approximately 20 cm long and is the same diameter as the base of the conical point. It is located immediately behind the point as shown in Figure 3-1. In practice, adhesion forces are measured as well as purely frictional forces. Viscous drag forces are limited to the extent possible via the quasi-static rate of penetration.

For granular, cohesionless soils, the magnitude of local friction encountered during a sounding is related primarily to the lateral effective stress[10] and the friction angle of the material. The greater the lateral effective stress and/or friction angle, the larger the value of local friction. In addition, the elasto-plastic behavior of the soil affects local friction measurements to the extent that the lateral effective stress is increased via cavity expansion upon penetration.

For fine-grained clayey (cohesive) soils, the measured local friction is a function of the undrained shear strength as well as frictional forces to the varying extent that the material behaves undrained (shear strength) versus drained (frictional). For a given point resistance, it is typically expected that a cohesive soil will yield higher local friction values as compared to a cohesionless soil. As for the case of point resistance, the value of local friction, in and of itself, cannot be used for definitive soil type identification.

3.2.3 Excess Pore Pressure

Measurement of the excess pore pressure generated as the piezocone penetrates the formation is a relatively recent development. Initially, the pore pressure transducer and porous stone system was placed in an 18° apex angle probe and used for measurement of steady state pore pressures only. This instrument, the piezoprobe (Wissa 1975), incorporated no provisions for measurement of point resistance or local friction. In fact, no effort was made to measure the instantaneous values of excess pore pressure generated as the probe was inserted to the depth of interest. The probe was therefore a reusable, very rapidly responding piezometer. The high stiffness value for the system allowed it to equilibrate with the formation very quickly and also minimized the change in formation pore pressure in the vicinity of the piezometer due to the act of measuring the pore pressure[11]. This characteristic made it ideal for measurement of rapidly varying pore pressures such as exist in clay formations subject to embankment loading.

---

[10] The lateral effective stress is that portion of the total lateral stress that is due to forces transmitted via inter-granular contact. Pore water fluid pressures are therefore not included in effective stress (Lambe, 1969, Pg. 241).

After initial problems with transducer overload from too fast an insertion rate in soft clays, pore pressure was monitored during insertion to protect the transducer. It was then noticed that the pore pressure generated during insertion was highly sensitive to material type, particularly sand senses in a clay formation or the converse. Piezoprobe capability was therefore expanded for use as an aid in stratification identification and then incorporated in the standard cone penetrometer (point and side resistance only) to form the piezocone.

The excess pore pressure generated due to constant rate penetration depends on three general soil characteristics. The first is the soils elasto-plastic response to cavity expansion, the second is the degree of dilatancy[12] exhibited by the soil in response to shear strain and the third is the soil's hydraulic conductivity. While the soil's behavior is actually a complex composite of the interrelationship between these effects, each will be treated as if independent in the following paragraphs to demonstrate general trends (Lexadoux 1980).

---

[11] Stiffness is a measure of the volume of water which must flow into/out of the measurement system for it to respond to a given change in formation pore pressure. The larger the volume required, the longer the equilibration time required. This is particularly critical in clayey formations where the low inherent hydraulic conductivity places severe limits on the rate of flow to the piezometer. This constraint not only applies to equilibration time following installation, but also to the time for the piezometer to respond to a change in formation pore pressure. If the volume required for system response is high (low stiffness) as compared to formation hydraulic conductivity, the response time may be greater than the duration of the change in formation pore pressure. As such, the act of measurement changes the parameter being measured and a true reading is never obtained.

[12] An increase in soil volume upon shearing due to the geometric interlocking of individual soil grains in dense soils (Lambe, 1969 Pg. 129).

As the piezocone instrument penetrates a soil mass, it compresses it due to volume displacement. The total stress therefore increases in the local vicinity of the sounding. The magnitude of the increase in total stress depends on the elasto-plastic response of the soil to the volume displacement (cavity expansion). An increase in the total stress tends to increase the pore pressure above the steady state value which existed prior to the sounding. The trend is therefore towards positive excess pore pressure generation for all soil types, but the magnitude is soil type dependent.

During penetration, the soil experiences severe shear strain as well as compression. Shear strain also tends to change the pore pressure. For this case however, not only is the magnitude of the change soil type dependent, but so is the direction of the change (positive vs. negative). The shearing process in loose sands and soft normally consolidated[13] clays tends to drive the excess pore pressure in a positive direction and thus complements compressional effects. As such, penetration of these materials generates a positive excess pore pressure which is superimposed on the previously existing steady state conditions. For dense sands and stiff overconsolidated[11] clays, dilatancy effects drive the shear related pore pressure in a negative direction. Hence, the shear component of the excess pore pressure opposes the compressional component. The total excess pore pressure can therefore be either positive or negative depending primarily on the degree of dilatancy exhibited by the soil (density of sands, degree of overconsolidation of clays).

As excess pore pressure is generated due to cavity expansion and shearing processes, it immediately begins to dissipate to the soil mass surrounding the sounding. The rate of drainage, as compared to the rate of penetration, directly affects the magnitude of excess pore pressure attained. Therefore, given that other parameters remain constant, a lower hydraulic conductivity soil (clays), will typically generate a higher excess pore pressure than a high hydraulic conductivity soil (sands). This is true for both positive and negative pore pressure generation.

---

[13] A normally consolidated soil is one which currently exists in equilibrium at a state of stress equivalent to the maximum stress state it has ever been subjected to. An overconsolidated soil currently exists at a state of stress less than that which has previously existed (Lambe 1969, pg. 74).

The rate of excess pore pressure dissipation is indicative of soil hydraulic conductivity. Low hydraulic conductivity soils drain slower than high hydraulic conductivity soils and thus the rate of excess pore pressure decay is slower. A relative indication of soil hydraulic conductivity can therefore be obtained by monitoring excess pore pressure decay when penetration is stopped. This is typically done when drill rod is added (at 5-foot increments). Figure 3-4 presents typical curves showing excess pore pressure and rate of decay for various soil types (Baligh, 1980).

3.2.4 Relative Hydraulic Conductivity Index

As indicated in Section 3.1.2, the standard piezocone was modified to include a permeameter for evaluating the relative hydraulic conductivity of the cutoff wall backfill. This technique had not been used prior to this project and, as such, expected response could not be determined based on past experience. The following discussion is therefore based on theoretical considerations, and is presented in the context of window identification.

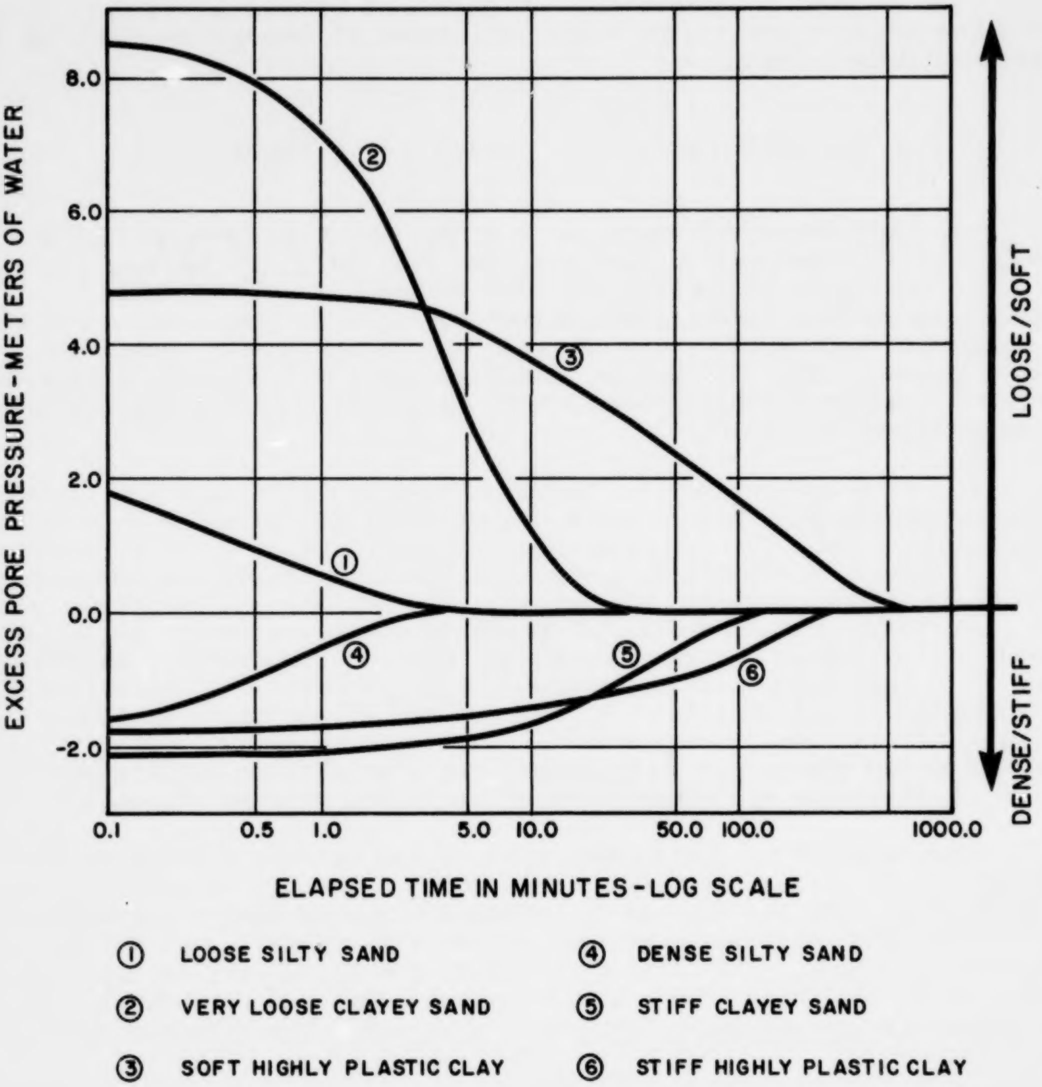
If the permeameter penetrates a material of constant hydraulic conductivity with depth, the water take[14] would be constant. This is because the driving pressure to the stone is maintained at a constant preset value above the formation pore pressure, independent of depth. For homogenous backfill placed as a vertical cutoff wall, it would be expected that the in-place backfill hydraulic conductivity would decrease somewhat with depth. The decrease in hydraulic conductivity results from increased consolidation stresses and thus density due to the weight of the overlying backfill. Therefore, water take would be expected to decrease with depth in the absence of windows. The rate of decrease with depth would be backfill type dependent.

The response obtained when penetrating a window depends primarily on three variables; 1) the hydraulic conductivity of the window, 2) the degree of communication between the window and the in-situ formation surrounding the cutoff wall and 3) the size of the window. As the hydraulic conductivity of the window increases, the water take would be expected to increase. If the window is in direct communication with a highly pervious

---

[14] Defined as the volume of flow into the formation per unit depth penetrated. Inasmuch as time is not incorporated in this correlation, the rate of penetration must be held constant.

FIG. 3-4 EXCESS PORE PRESSURE DISSIPATION
(WISSA, 1975)



formation surrounding the cutoff wall, then the water taken would increase in proportion to the window hydraulic conductivity (within the constraints that the hydraulic conductivity of the window remains low as compared to the formation and the flow required does not cause significant head loss in the permeameter system). However, if the window is totally contained within the backfill, then water take will be limited by the rate at which water can flow out of the window into the surrounding backfill. In this case, water taken will increase in proportion to the surface area of the window and thus window size (within the constraints that the backfill hydraulic conductivity is low in comparison to that of the window).

3.2.5 Empirical Correlations

Empirical correlations have been developed between piezocone response and soil type. These correlations are typically based on the relationship between a "secondary" index (local friction or excess pore pressure) and the primary index of point resistance. These relationships are generally expressed in a normalized fashion.

The first correlation to be developed was between local friction and point resistance. In this case, the value of local friction was normalized with respect to point resistance resulting in the friction ratio index (computed as the local friction divided by point resistance and reported in percent). Figures 3-5 and 3-6 present two such correlations which demonstrate that: 1) for a given point resistance, increasing friction ratio indicates a finer grained, more cohesive soil and 2) an increasing point resistance at a given friction ratio, indicates a coarser grained more cohesionless soil.

The second major correlation of interest is between the excess pore pressure generated during a sounding and point resistance. One such example is presented in Figure 3-7. In this case, excess pore pressure has been normalized with respect to steady state hydrostatic pore pressure and the point resistance has been normalized with respect to the total vertical stress. This correlation demonstrates that 1) for a given point resistance, increasing excess pore pressure generation indicates a finer grained, more cohesive soil and 2) an increasing point resistance, for a given excess pore pressure, indicates a coarser grained, more cohesionless soil.

The above correlations can be used to define soil type on a general basis, only. The standard correlations are typically

FIG. 3-5 MATERIAL IDENTIFICATION- FRICTION RATIO (DOUGLAS, 1981)

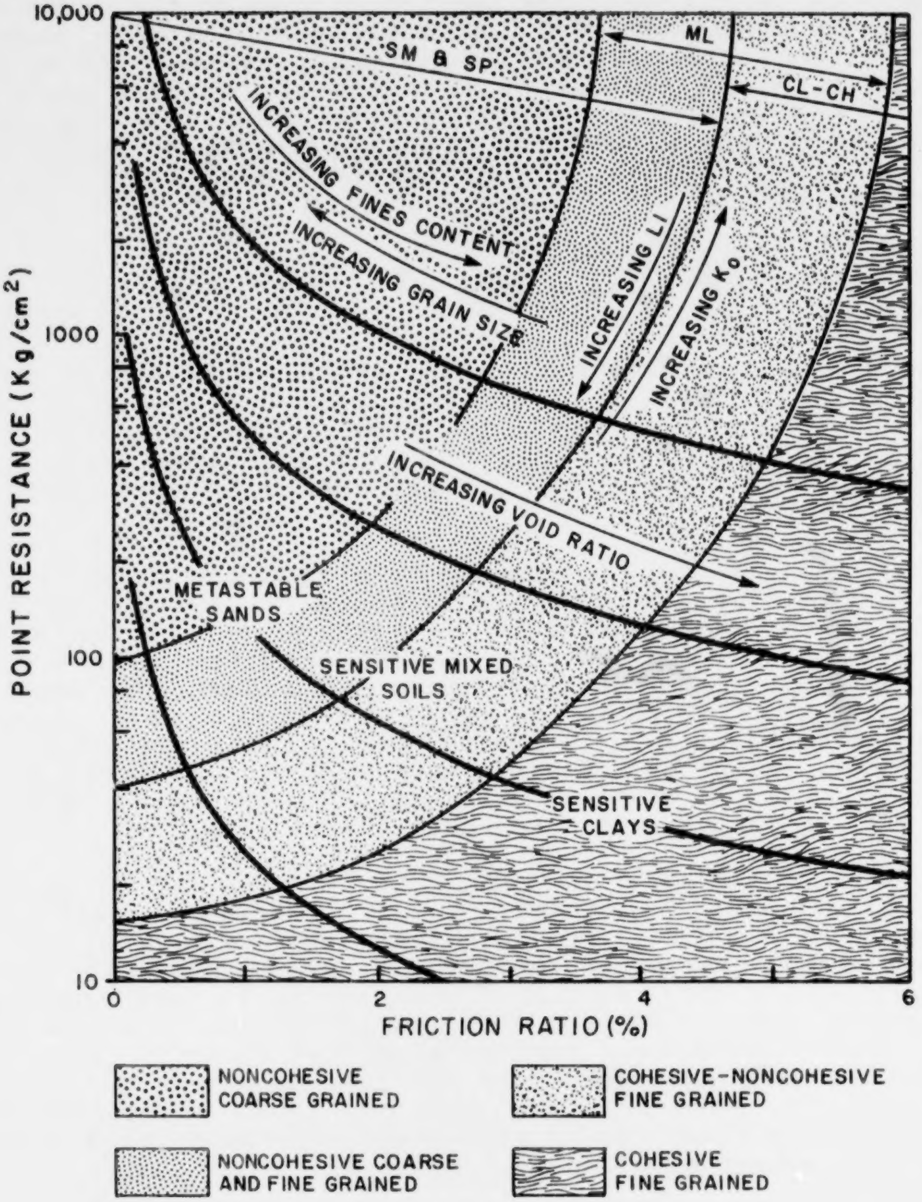
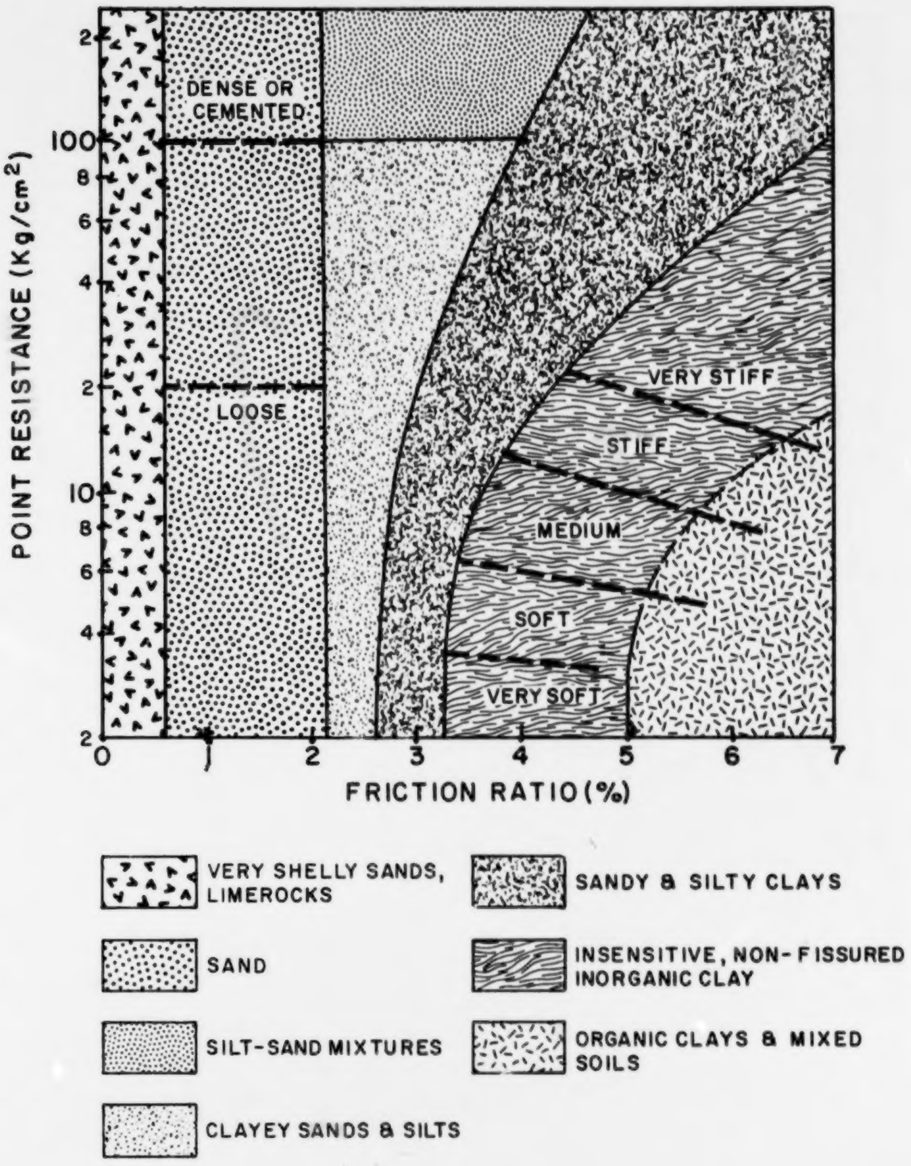
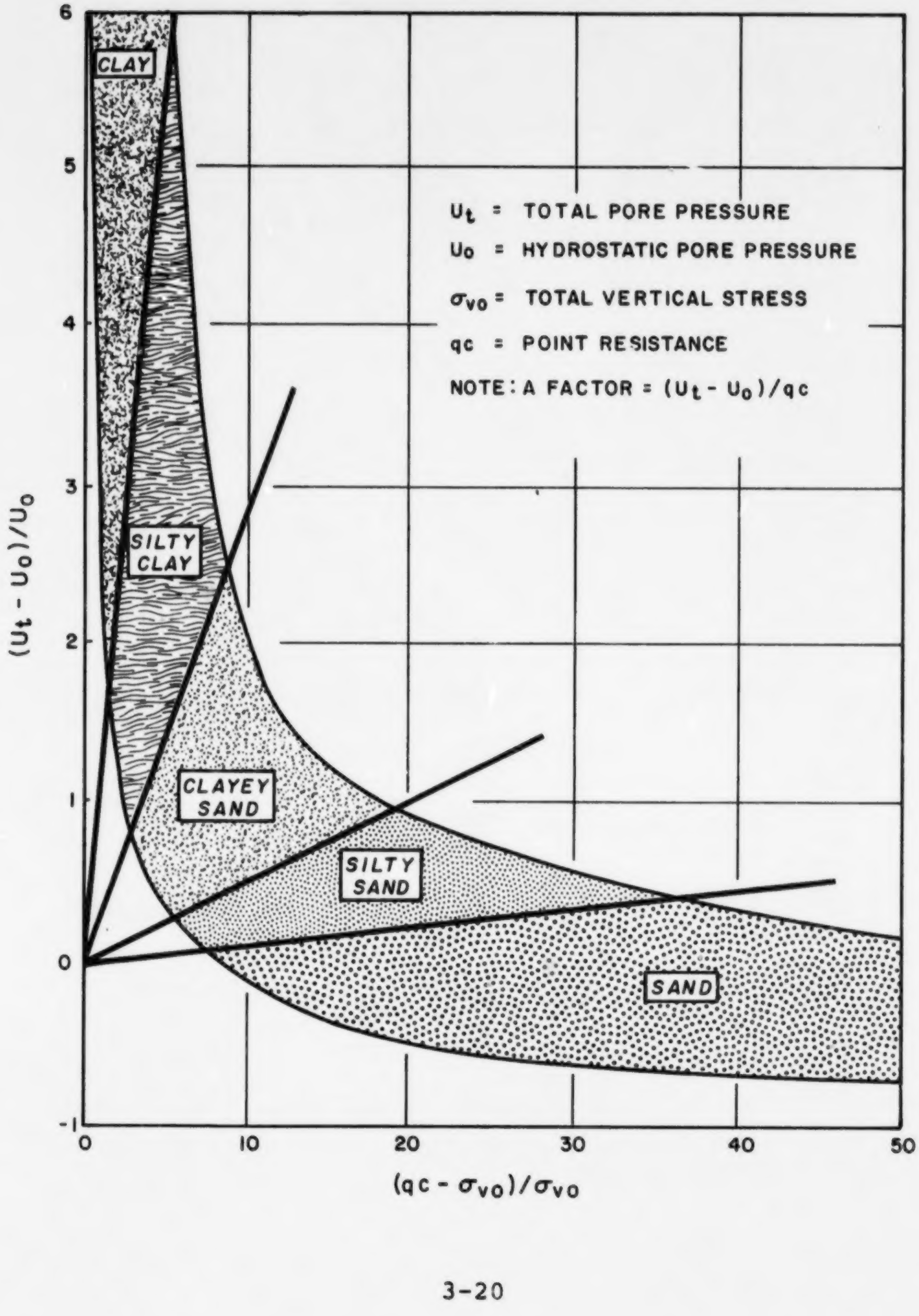


FIG. 3-6 MATERIAL IDENTIFICATION-FRICTION RATIO (SCHMERTMANN, 1977)



NOTE: BASED ON CORRELATIONS DEVELOPED FOR THE NORTH CENTRAL FLORIDA GEOLOGY.

FIG. 3-7 MATERIAL IDENTIFICATION-EXCESS PORE PRESSURE (JONES, VANZYL, 1981)



modified based on empirical data generated for a local's specific geology. Hence the soil identification curves can vary significantly as demonstrated by Figures 3-5 and 3-6, particularly at low point resistance. Therefore, caution must be exercised when applying the general correlations to deposits for which no prior data exists.

3.2.6 Window Identification

The occurrence of windows in completed cutoff walls and their characterization is poorly documented. As such, not only is there a lack of empirical correlations for window identification, but the composition of actual windows has not been investigated. Therefore, the following discussion of expected piezocone response is based on the probable composition and behavior of windows as based on hypothesized mechanisms leading to their formation. An understanding of the response to be expected, as based on theoretical considerations, is critical to the interpretation of field data obtained during sounding of the test section and the Gilson Road containment wall as presented in subsequent sections.

The three most likely types of windows of concern are:

1. Clean granular window - Composed of either 1) unexcavated in-situ formation soils, or 2) on-site formation soils which sloughed off the trench wall, remained intact, and became entrapped in the backfill. In the first instance, it is expected that the material would remain unaltered with respect to the parent formation and thus retain it's density and strength. In the second case, it is anticipated that the material would show a decrease in density and strength. In either case, the window would contain original groundwater as the pore fluid.

2. Slurry-filled granular window - Composed of on-site formation soils entrained in the slurry during excavation which sediment out and contain bentonite support slurry as the pore fluid.

3. Bentonite slurry window - Composed of bentonite support slurry entrapped during slurry displacement by the backfill.

For each of the above windows, the expected response for the four piezocone transducers will be discussed individually in the following sections. This is preceded by a discussion of expected response for intact soil/bentonite backfill. Figure 3-8 presents a schematic representation of the expected relative piezocone response to backfill and windows.

3.2.6.1 Intact Soil/Bentonite Backfill

The soil/bentonite backfill specified for the Gilson Road site consisted of a primarily sand and gravel mixture augmented to contain not less than 30 percent fines and not less than 5 percent bentonite. The material was blended and wetted with trench support slurry until it reached a slump of 4 to 6 inches[15]. This plastic material was placed in the trench with a consistency of toothpaste and it consolidated with time to a soft to medium consistency[16].

A. Point Resistance

The soil/bentonite backfill used at the Gilson Road site was far less dense than the existing in-situ granular formations. Point resistance should therefore be relatively low. It was also expected that the material would behave as a plastic, rather than granular, matrix further reducing expected point resistance.

With depth, the backfill is subjected to greater stresses and point resistance was therefore expected to increase. However, it should still remain well below values associated with the in-situ formation. Localized, higher point resistance "spikes", superimposed on this general trend, would be anticipated due to cobbles and gravel in the plastic soil matrix.

B. Local Friction

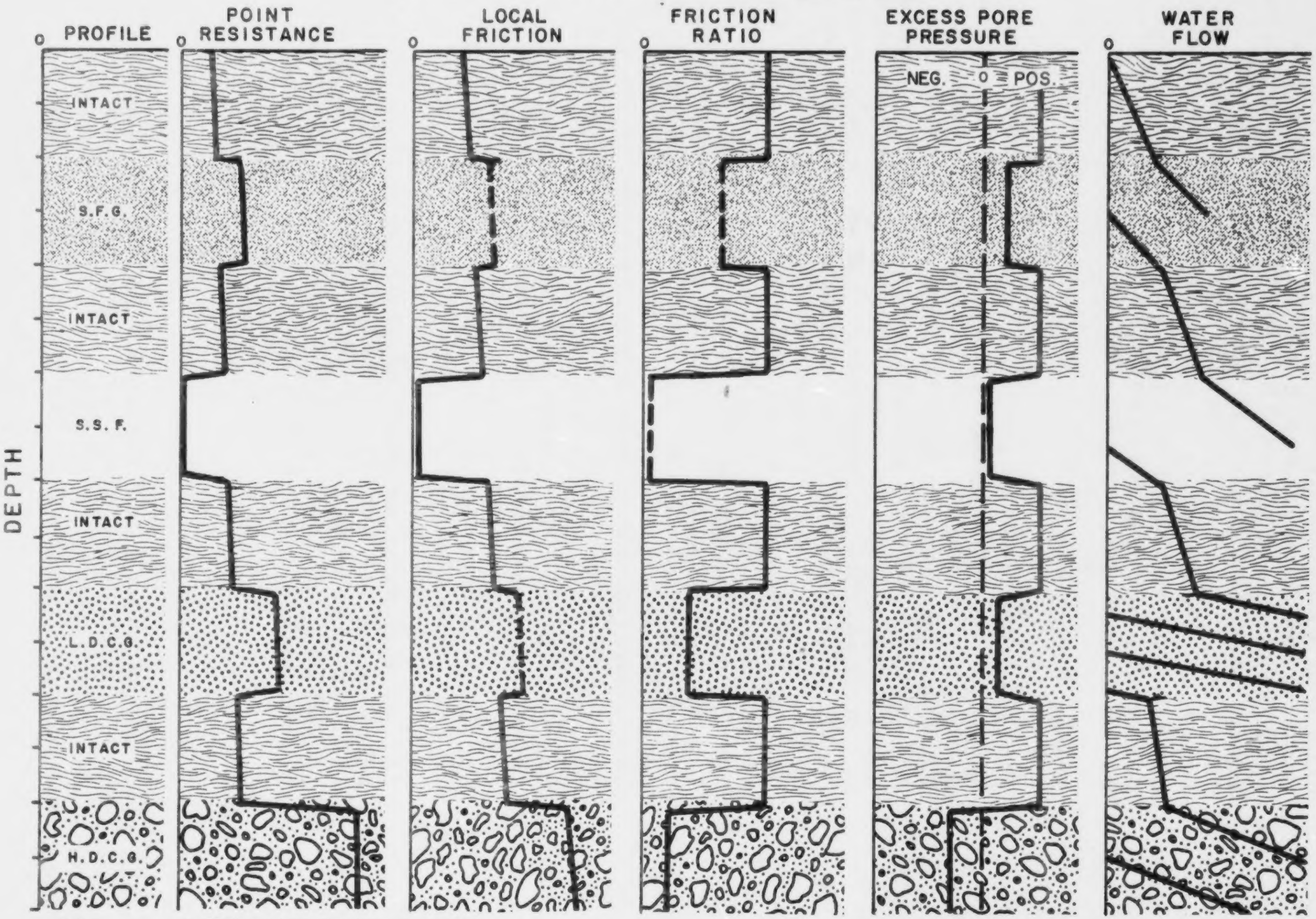
The absolute value of local friction attributable to the soil/bentonite backfill is again expected to be much lower than that of the existing in-situ formation. Although the backfill is plastic, thus indicating relatively high local friction due to adhesion characteristics, the much greater density of the in-situ granular formation results in greater frictional forces.

---

[15] Slump was measured with a standard concrete slump cone in accordance with ASTM Standard C 143-78.

[16] The consistency of plastic soil deposits are standardly rated from very soft to hard in accordance with data presented in Lambe 1969, pg. 72.

FIG. 3-8 RELATIVE PIEZOCONE RESPONSE


PROFILE
POINT RESISTANCE
LOCAL FRICTION
FRICTION RATIO
EXCESS PORE PRESSURE
WATER FLOW
0
NEG.
POS.
DEPTH
INTACT
S.F.G.
INTACT
S.S.F.
INTACT
L.D.C.G.
INTACT
H.D.C.G.

S.F.G. - SLURRY FILLED GRANULAR WINDOW
S.S.F. - SUPPORT SLURRY FILLED WINDOW
L.D.C.G. - LOW DENSITY CLEAN GRANULAR WINDOW
H.D.C.G. - HIGH DENSITY CLEAN GRANULAR WINDOW

With depth, local friction should increase due to the associated higher state of stress and thus higher undrained shear strength. Although the magnitude of local shear is expected to be less for the backfill than for the in-situ granular formation, the value of friction ratio should be more consistent with the plastic nature of the backfill. Hence, the friction ratio would be anticipated to be greater for the backfill than the granular native formation.

C. Excess Pore Pressure

The soil/bentonite backfill is both almost completely granular and relatively plastic at the same time. As such, it is atypical of most natural soil deposits. This behavior is due to the exceptionally high activity[17] of the bentonite component. However, by separating the two characteristics, it can be shown that the excess pore pressure generated while penetrating the backfill should generally be positive. If a completely cohesionless soil existed at the same density as the backfill, it would be considered loose. As such it would not exhibit dilatant behavior and thus would generate positive excess pore pressures. A totally cohesive material mixed and placed under the same conditions as the backfill, would exist under normally consolidated conditions. Again, this would indicate that positive excess pore pressures should be expected. While the prediction of positive excess pore pressure generation is straight forward as based on theoretical considerations, the magnitude of the expected pressures cannot be quantitatively defined. However, the excess pore pressures generated should be greater for the backfill as compared to the formations existing in-situ at the Gilson Road site.

If the sounding is stopped, such as during drill string addition, it is expected that the excess pore pressure will dissipate at a slow rate. The rate of dissipation should be consistent with the low hydraulic conductivity of the backfill (1×10^{-7} cm/sec).

---

[17] Activity is a standard soil mechanics parameter which indicates the degree of plasticity a given soil component (typically clay sizes) will impart to a soil mixture, as related to the percentage of that component in the soil; Lambe, 1969, pg. 34.

D. Relative Hydraulic Conductivity

By definition, the water take in the backfill is expected to be consistent with the backfill hydraulic conductivity (1×10^{-7} cm/sec). Given the backfill hydraulic conductivity, the applied driving pressure, and the appropriate geometries, it is possible to determine, quantitatively, the expected flow rate. However, this analysis was not performed in that the scope of the project was directed toward investigation of the suitability of this index for window detection in the cutoff wall. As such, the effort was limited to establishing relative values of water take.

3.2.6.2 Clean Granular Window

Material composing a clean granular window would retain the original formation gradation. The hydraulic conductivity and strength of the window would be dependent on the mechanism leading to its formation. However, the magnitude of each of these parameters is expected to remain significantly greater than exhibited by the intact backfill.

A. Point Resistance

Point resistance is proportional to the strength of the material penetrated. Therefore, it is expected to show an increase when penetrating a clean granular window. The magnitude of the increase is dependent on whether the window is of low density, such as that formed by material sloughing off the trench side wall and falling onto the backfill, or high density, as would be expected for unexcavated material.

B. Local Friction

The decreased adhesion and coarse grained nature of the window, as compared to the backfill, would tend to decrease the value of local friction. However, the increased strength of the window material would increase the value of local friction. Therefore, the direction of the change in local friction cannot be predicted. However, the friction ratio (local friction divided by point resistance) would be expected to decrease when encountering the window.

C. Excess Pore Pressure

To the extent that the hydraulic conductivity of the in-situ materials are preserved in the window, it would be expected that excess pore pressure would decrease when entering the window. The magnitude of the decrease should be dependent on the dilatant nature of the materials. Therefore, the decrease in excess pore pressure should be greater for high density windows than for low density windows.

D. Relative Hydraulic Conductivity

Water take would typically be expected to show a marked increase when the sounding penetrates the window. This is directly due to the large increase in hydraulic conductivity for the clean granular materials as compared to the backfill (10^{-7} cm/sec). If a significant portion of the window is in direct communication with the in-situ formation (no separation by intact backfill or filter cake), then the water take will be limited only by head loss within the permeameter system. This is a direct result of the high hydraulic conductivity associated with the in-situ deposits at the site. However, if the window is totally encapsulated within intact backfill, then the actual magnitude of water take will be a function of the size of the window. In this case, the water take is limited by the flux at the window/backfill interface. Therefore, the window becomes analogous to having a porous permeameter stone with an area equal to that of the window. This is attributable to the disparity between window and backfill hydraulic conductivities. Even in the case of a totally encapsulated window, water take is expected to increase significantly as compared to the intact backfill[18].

Based on the above discussion, it is anticipated that a clean, granular window would most easily be discriminated using the parameters of point resistance and relative hydraulic conductivity. Both indexes are expected to exhibit significant increases in magnitude when the sounding encounters a window of this type.

---

[18] The surface area of even a small window 1 Ft^2 in dimension (1,728 in^2) is over four orders of magnitude greater than the permeameter stone area (3.6 in^2).

3.2.6.3 Slurry Filled Granular Window

The sedimentation mechanism leading to the formation of this type of window should result in uniformly graded (poorly sorted), low density material with support slurry filling the void spaces. The actual grain size of the material comprising the window would be dependent on the gradation of the in-situ materials being excavated, the viscosity of the support slurry, and the time period between excavation and backfilling.

A. Point Resistance

Due to the low density of the window, it is anticipated that point resistance will be low. However, inasmuch as the resistance to penetration in the backfill is also inherently low, the point resistance may show an increase relative to intact backfill. The magnitude of point resistance in such a window is expected to be highly grain size dependent. (Slurry deposited fine sand would be expected to exhibit a very loose structure, where as a gravel filled window would be relatively difficult to penetrate.) Therefore, the point resistance for the window may actually be much greater than that for the backfill in specific instances.

B. Local Friction

The slurry filled granular window is expected to be an essentially granular deposit. However, the slurry filling the voids may impart significant adhesion, depending on its consistency and thixotropy[19]. As such, local friction could not be predicted. As indicated above, point resistance, as well as local friction, cannot be predicted. Therefore, friction ratio can not be predicted.

C. Excess Pore Pressure

The low density of the material should preclude dilatant behavior. Therefore, it is anticipated that a positive excess pore pressure would be generated when sounding through a slurry filled granular window. The relative magnitude of the excess pressure compared to that for the backfill, as well as the absolute magnitude, is indeterminate with the existing theoretical understanding.

---

[19] Strength increase with time at constant composition and stress (Lambe, 1969, Pg. 458).

D. Relative Hydraulic Conductivity

It is expected that the water take for this type of window would be similar to that for a support slurry filled window (see Section 3.2.6.4). However, for a given viscosity slurry in the window, it would be expected that the water take for the slurry filled granular window would be somewhat less than the support slurry filled window. This results from the added resistance to flow commensurate with the increase tortuosity associated with the small intergranular pore size. In any case, the water take is still expected to be greater in the window than in the intact backfill.

Based on the above criteria, it is expected that a slurry filled granular window will be the most difficult to detect of the three types of windows discussed. However, this type of window is also expected to have a relatively low hydraulic conductivity, high stability, and thus the least detrimental affect on the overall efficiency of the cutoff wall; except in special cases. (If the window is filled with gravel size material and is adjacent to a very coarse in-situ deposit, a high gradient can remove the slurry via piping[20]. This will leave a very high hydraulic conductivity window which totally breaches the barrier.)

3.2.6.4 Bentonite Support Slurry Filled Window

The window in this case is composed of the bentonite support slurry which has become entrapped by the advancing backfill. It is hypothesized that this condition could result if the slurry is allowed to become too heavy and is overridden rather than displaced by the backfill.

A. Point Resistance

Inasmuch as the window is a fluid, point resistance is expected to be zero. However, a small point resistance may be measured due to the thixotropic behavior of the bentonite slurry.

---

[20] For a definition and mechanisms leading to piping, the reader is referred to Middlebrooks, (1953), and Cedergren, (1967).

B. Local Friction

Local friction values within a fluid are also zero. A small value may be measured due to viscous drag forces if the slurry is very thick. Adhesion forces may also develop due to thixotropic effects.

C. Excess Pore Pressure

Assuming the backfill has totally consolidated and the slurry window remains intact, the fluid pressure in the window will be consistent with hydrostatic conditions. Inasmuch as the slurry is a fluid, shear generated excess pore pressure would be zero. Therefore, excess pore pressure generation would be consistent with volumetric displacement of the slurry as the probe penetrates the window and therefore should be positive. The magnitude of the excess pore pressure would be dependent on the rate at which the "compressed" slurry could permeate out of the window. Factors influencing this behavior include surface area of the window and thus its size, viscosity of the slurry, and hydraulic conductivity of the sides of the window (either soil/bentonite backfill, slurry trench filter cake and/or natural formation).

D. Relative Hydraulic Conductivity

The water take would typically be expected to increase when penetrating a slurry filled window from an intact backfill. The water introduced into the window would pressurize the slurry and cause the slurry to permeate into the backfill. While the viscosity of the slurry as a permeant is greater than that of water, it is likely that the increased surface area (of the window as compared to permeameter stone in direct contact with backfill) would more than offset this factor.

Based on the above discussion, it is expected that the most definitive indicators of a slurry filled window would be severely reduced point resistance and local shear as compared to intact backfill.

3.3 Cutoff Wall Test Section

While the standard piezocone has been widely used for stratigraphic investigation over the past 10 years, its technical and economic feasibility has not been demonstrated for cutoff wall anomaly detection. To evaluate technical feasibility, a full scale cutoff wall test section was constructed. The objective of test section construction was to incorporate windows of known type and location such that the response of the instrument, when probing a known condition, could be evaluated.

3.3.1 Test Section Construction

The test section was constructed in the northwest corner of the Gilson Road containment (see Figure 3-9) and was approximately 3.5 feet wide by 24 feet long by 21 feet deep. Construction followed the same methodology as used during construction of the containment cutoff wall with the exception that windows were purposely incorporated into the backfill.

3.3.1.1 Excavation

Prior to excavation, the bentonite support slurry was mixed using Baroid National Premium sodium bentonite, a 90 barrel yield Premium Grade[21] bentonite (as specified for construction of the Gilson Road containment wall). The mixing proportions were 7 percent bentonite in water, by weight. The water used was drinking water quality spring water. The support slurry was placed in a holding pond after mixing to allow complete hydration. Marsh funnel viscosity and unit weight quality control data are presented in Table 3-1.

---

[21] In accordance with testing standards developed by the American Petroleum Industry, Method 13A and B (API, 1981 and API, 1982).

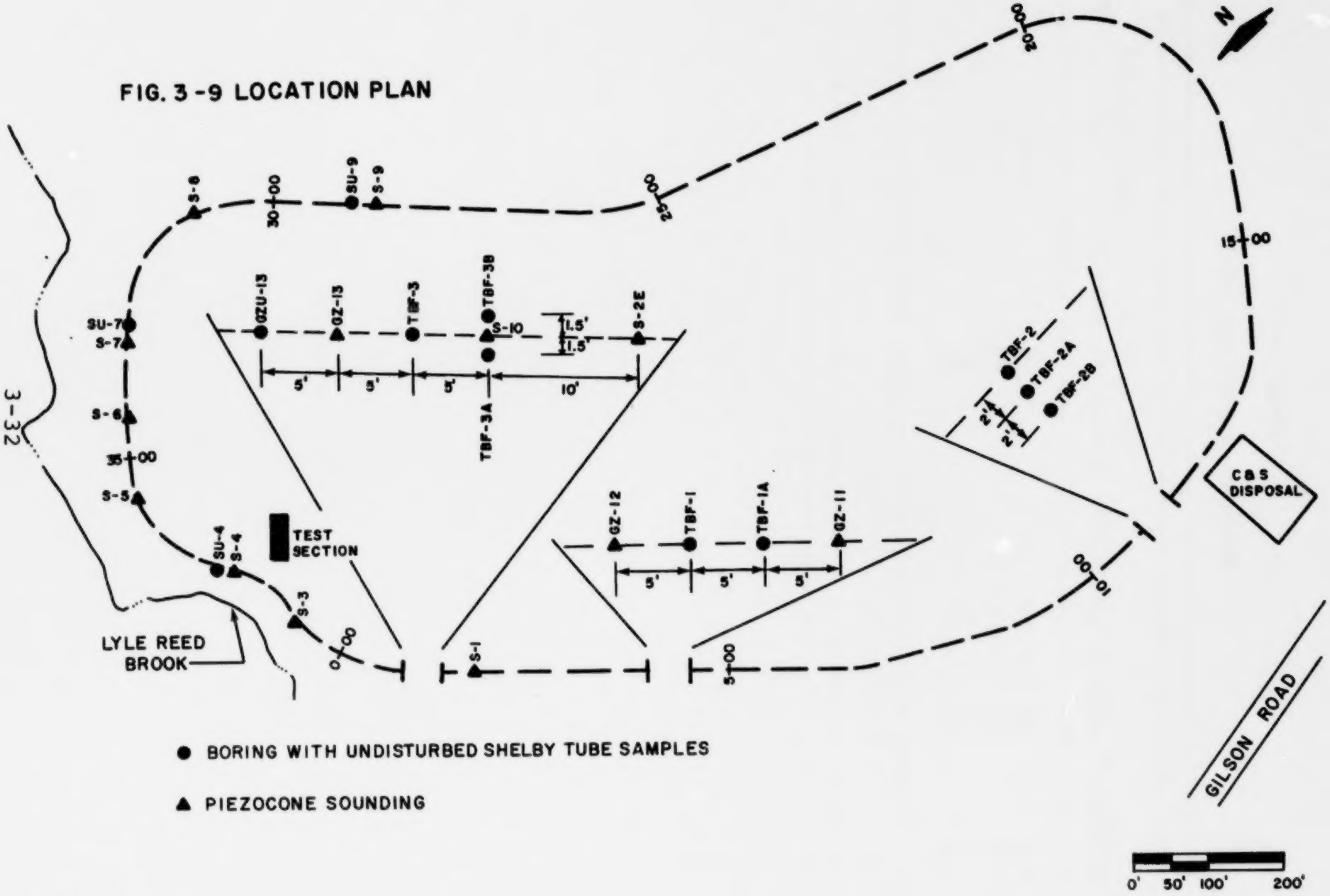
TABLE 3-1

SUPPORT SLURRY QC DATA

Sample #	Sample Location	Marsh Funnel Viscosity (sec.)	Unit Weight (PCF)	Notes
1	SE Corner	38	65.5	1
2	NE Corner	42	--	1
3	NW Corner	37	--	1
4	SW Corner	45	66.5	1

1. Sample taken from hydration pond just prior to excavation.
2. -- indicates not tested.

FIG. 3-9 LOCATION PLAN


N
0+00
5+00
10+00
15+00
20+00
25+00
30+00
35+00
S-1
S-3
S-4
SU-4
S-5
S-6
S-7
SU-7
S-8
SU-9
S-9
GZU-13
GZ-13
TBF-3
TBF-3B
S-10
TBF-3A
S-2E
1.5'
1.5'
5'
5'
5'
10'
GZ-12
TBF-1
TBF-1A
GZ-11
5'
5'
5'
TBF-2
TBF-2A
TBF-2B
2'
2'
C&S DISPOSAL
GILSON ROAD
TEST SECTION
LYLE REED BROOK
● BORING WITH UNDISTURBED SHELBY TUBE SAMPLES
▲ PIEZOCONE SOUNDING
0'
50'
100'
200'

Trench excavation was completed under slurry stabilization. The trench remained open for only one hour between excavation and backfilling. Therefore, full filter cake development was unlikely.

3.3.1.2 Backfilling

The backfill was composed of a mixture of the excavated sand and gravel (trench spoil), imported sandy silt borrow, dry powdered bentonite and support slurry in the same proportions used for the Gilson Road containment. The mixture was blended via bulldozer until it reached a uniform consistency, as determined visually. Quality control testing data is presented in Table 3-2.

TABLE 3-2

SOIL/BENTONITE BACKFILL QC DATA

Sample No.	Slump[1] (inches)	Unit Weight (PCF)	Fines[2] Content (percent)	Bentonite[3] Content (percent)	Hydraulic[4] Conductivity (cm/sec)	Notes
1	5.5	123.2	24	2.8	1.7×10^{-7}	5,6
2	5.5	124.0	22	2.6	2.0×10^{-7}	5,7
3	5.0	125.0	25	2.4	2.0×10^{-7}	5,8

1. ASTM Method C-143-78
2. Percent passing number 200 sieve
3. Determined using methylene blue titration
4. Determined using API Fixed Ring Test
5. Samples taken prior to backfill placement
6. Placed in bottom 3 foot of trench
7. Placed between 10 and 15 foot depth
8. Placed at top of trench

The backfill was placed in the trench one bucket at a time. The backhoe lowered the filled bucket into the slurry to the top of the backfill surface. The bucket was then slowly rotated to deposit the backfill. This method was similar to backfill placement at the beginning of actual containment construction (initial backfill placement until it rose above slurry surface). However, it does not simulate the lateral movement characteristic of production backfill placement during most of the Gilson Road containment wall construction. The associated backfill placement techniques could not be simulated on the small scale of the test section wall.

3.3.1.3 Window Emplacement

Four different types of windows were emplaced in the test section. These were: 1) a high density clean granular window, 2) a low density clean granular window, 3) a slurry filled granular window, and 4) support slurry filled windows. The design location and dimensions of these windows is depicted in Figure 3-10. These dimensions were verified by measurements along the trench axis with a weighted tape after placement of each layer.

It is emphasized that the mechanisms leading to window formation and their composition has not been previously documented. The objective of this effort was therefore to simulate windows which were judged likely to occur during actual cutoff wall construction as based on previous field observations and theoretical considerations. These windows were constructed using the following techniques.

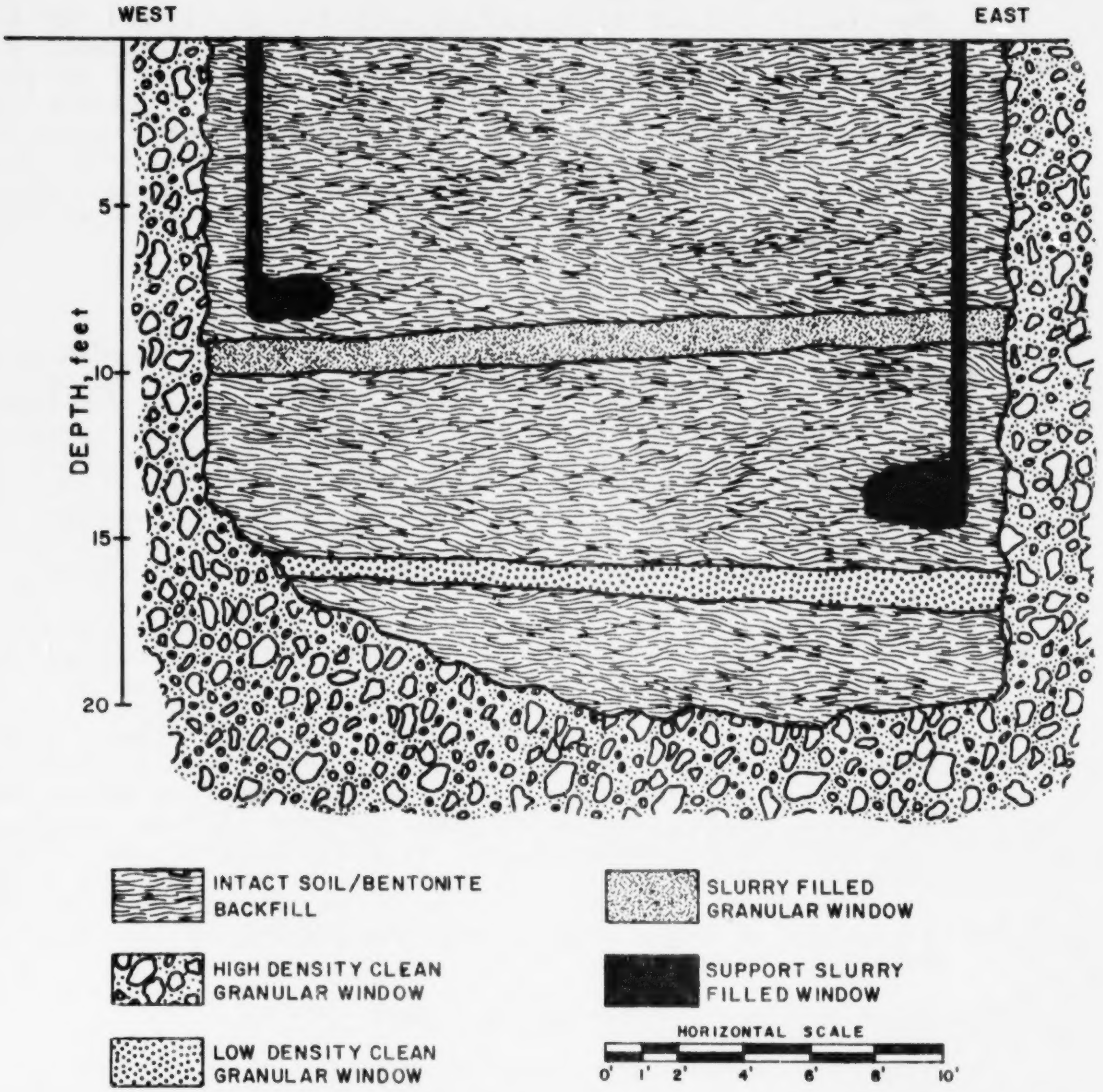
A. High density clean granular window (HDCG)

This window was meant to simulate the characteristics of clean granular unexcavated material (Section 3.2.6). The high density was required to be consistent with the character of the in-situ formations encountered on-site. This window was the easiest to create, inasmuch as the unexcavated material below the bottom of the trench matches this hypothesized type of window perfectly. The materials comprising the window were in direct hydraulic communication with the in-situ granular materials on-site. The profile of the trench bottom and thus the top of this window is shown on Figure 3-10.

B. Low Density Clean Granular Window (LDCG)

A low density counterpart to the window described above was also emplaced in the test section. The objective in this case was to simulate a window formed due to sloughing of intact portions of the trench side wall

FIG. 3-10 TEST SECTION CONSTRUCTION PROFILE


WEST
EAST
DEPTH, feet
5
10
15
20
INTACT SOIL/BENTONITE BACKFILL
HIGH DENSITY CLEAN GRANULAR WINDOW
LOW DENSITY CLEAN GRANULAR WINDOW
SLURRY FILLED GRANULAR WINDOW
SUPPORT SLURRY FILLED WINDOW
HORIZONTAL SCALE
0'
1'
2'
4'
6'
8'
10'

NOTE: HIGH DENSITY CLEAN GRANULAR WINDOW IS MATERIAL IN WHICH TRENCH WAS EXCAVATED.

which retained the existing pore fluid but not the existing density. This window would be hydraulically isolated from the in-situ parent formation by the filter cake.

The window was constructed after the trench was partially filled with soil/bentonite backfill by 1) placing concrete sand in the backhoe bucket, 2) saturating the sand with water, 3) lowering it to the surface of the in place backfill, and 4) rotating the bucket to deposit the sand. This process was repeated until a layer of material, as shown in Figure 3-10, was created. Subsequent to sand placement, additional soil/bentonite backfill was placed over this window using the same placement technique.

C. Slurry Filled Granular Window (SFG)

The slurry filled granular window was meant to simulate a window formed via the sedimentation of coarse material out of the slurry (see Section 3.2.6 for further description). In this case, concrete sand was dropped into the top of the slurry filled trench and allowed to settle to the top of the previously placed soil/bentonite backfill. After an appropriate volume of material was placed and allowed to settle, additional soil/bentonite backfill was placed as described above.

It is also likely that a similar slurry filled window incorporating the in-situ granular formation material was, to varying extents, created at the bottom of the trench. In all cases, a filter cake of bentonite slurry would form at the trench bottom. However, this filter cake would only be about 1/4-inch thick. A much thicker slurry filled granular window would be created where the backhoe teeth scarified the trench bottom during excavation and thus mixed support slurry with those deposits. Where this occurred, the window may be 6 to 12 inches thick in vertical dimension. The exact locations of these areas along the bottom of the trench were not known.

D. Support Slurry Filled Window (SSF)

The slurry filled window was installed after the test section was completely backfilled. Although this format does not chronologically correspond to that anticipated during actual containment construction, it was the only way window location could be controlled.

Two slurry filled windows were constructed, one at each end of the test section (see Figure 3-10). The installation procedure consisted of driving a casing to the depth of the window and then washing out the casing. A side ported washing bit was then inserted into the backfill below the casing. Slurry was pumped out of the bit under high pressure to erode the intact backfill. The slurry, carrying the cuttings, was brought to the surface through the annular space between the jetting tool and the casing. The slurry was recirculated through a sedimentation tub and the cuttings removed. The cuttings consisted primarily of coarse to fine sand and silt with about 10 percent fine gravel. The gravelly backfill fractions could not be suspended in the return slurry and were thus left at the bottom of the cavity.

The vertical thickness of the windows ranged between 12 and 18 inches as determined by measurement below the casing. The cavities were expanded until no further cuttings were removed by the slurry. The lateral extent of the cavities, as shown on Figure 3-10, was estimated by evaluating the erosional capabilities of the slurry jet in the backfill at the surface of the test section.

3.3.2 Test Section Window Verification

As previously stated, the objective of the test section was to provide a known condition with which to evaluate piezocone response. As such, numerous measurements were taken before and after each lift of material was placed. These data were used to generate Figure 3-10. However, once a lift was placed, there was no method available to ensure that it remained in position as the subsequent lifts were placed on top. Displacement of previously placed lifts was possible due to:

- Inability of the backhoe operator to "feel" the material as he rotated the bucket to add new material.

- Low unit weight, low stability materials (slurry filled granular window) placed below higher unit weight backfill.

- Unknown long-term stability of the window (support slurry filled window).

Ideally, an independent verification method would be used to document actual in-place conditions in the test section after it had stabilized. Such verification methods would include continuous undisturbed sampling at close spacings along the trench alignment or complete excavation. While these methods are feasible if performed after piezocone sounding, the associated costs placed them outside the scope of this project.

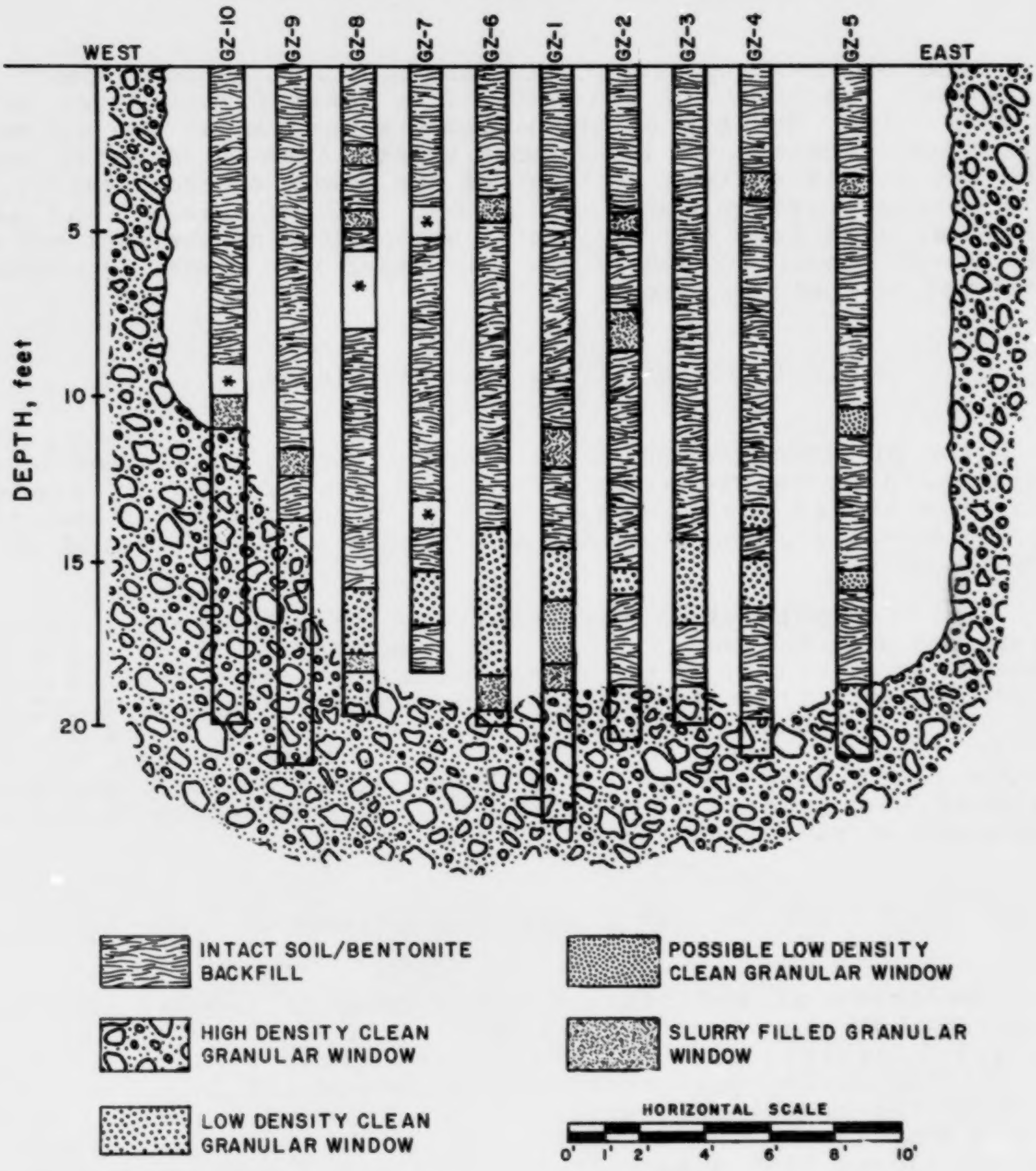
To the extent that the piezocone response to intact backfill and windows is unknown, the sounding data cannot be used to verify window location. In fact, it is the piezocone response which was being investigated. However, while piezocones have not previously been employed to investigate soil/bentonite backfilled cutoff walls, theoretical considerations and empirical data from other applications provided insight into expected response (Section 3.2.6). The piezocone data was therefore used to verify window location as based on expected trends.

The piezocone sounding data was analyzed, correlated, and then used to construct a post-construction, as-built test section profile. (The individual soundings and window types are discussed in detail in Section 3.3.3.) This profile, presented in Figure 3-11, reveals some significant discrepancies when compared to the profile developed from construction measurements (Figure 3-10). These discrepancies are discussed individually in the following sections along with potential explanations for the deviations.

3.3.2.1 High Density Clean Granular Window

The high density clean granular, H.D.C.G., window was simulated by the unexcavated materials below the trench bottom. As such, the backfill/window interface was the trench bottom. The only discrepancy found in this case, existed at the extreme west end of the trench. It appears that the initially vertical west end of the trench may have sloughed off or been dragged down by the backhoe bucket. The material thus raised the elevation of the trench bottom. This hypothesis is supported by point resistance data from soundings GZ-9 and GZ-10. In both cases, the magnitude of point resistance nearly doubles at depths corresponding to the predicted trench bottom. Another plausible explanation is that the trench was not excavated to the full depth. However, this is not supported by the measurements taken during construction.

FIG. 3-11 POST-CONSTRUCTION TEST SECTION PROFILE


WEST
GZ-10
GZ-9
GZ-8
GZ-7
GZ-6
GZ-1
GZ-2
GZ-3
GZ-4
GZ-5
EAST
DEPTH, feet
5
10
15
20
*
INTACT SOIL/BENTONITE BACKFILL
HIGH DENSITY CLEAN GRANULAR WINDOW
LOW DENSITY CLEAN GRANULAR WINDOW
POSSIBLE LOW DENSITY CLEAN GRANULAR WINDOW
SLURRY FILLED GRANULAR WINDOW
HORIZONTAL SCALE
0'
1'
2'
4'
6'
8'
10'

NOTE 1. HIGH DENSITY CLEAN GRANULAR WINDOW IS MATERIAL IN WHICH TRENCH WAS EXCAVATED.

2. * INDICATES NO DATA

3.3.2.2 Low Density Clean Granular Window

The low density clean granular, L.D.C.G., window appears to have been successfully constructed in general accordance with Figure 3-10. The top of the window is located at the correct elevation. However, it is somewhat thicker than anticipated over the center and eastern portions of the trench and nonexistent at the extreme west end of the trench. It is likely that the material slid down the apparently steep slope at the west end of the trench (Section 3.3.2.1) and increased the window thickness over the rest of the trench.

3.3.2.3 Slurry Filled Granular Window

The piezocone soundings did not detect a consistent layer corresponding to the slurry filled granular, S.F.G. window anywhere within the trench profile. The data indicated that the window material probably exists as discrete pockets located in a random manner.

It is hypothesized that the layer of window material was displaced when the backfill was placed on top of it. The S.F.G. window would be more prone to instability than the L.D.C.G. window because 1) it had a lower unit weight than the L.D.C.G. window (probably also lower than the backfill) and 2) the material would behave in a more cohesive manner due to the bentonite slurry in the voids. The second property may have allowed the material to behave as a viscous liquid during placement of subsequent backfill lifts.

3.3.2.4 Support Slurry Filled Window

Behavior of undisturbed soil/bentonite samples from the completed Gilson Road containment wall indicated that the material exhibited significant strength. This was further documented during post-construction excavation into the top 5 feet of the containment wall. In this case, the backfill would easily support a person's weight. Based on these observations, it was judged that a support slurry filled, S.S.F., window could exist in the containment wall and thus could probably be constructed in the test section. As described in Section 3.3.1.3, two such windows were constructed in the test section. Measurements made during window installation verified that the slurry filled cavities were, in fact, created.

Upon completion of the soundings, however, it was evident that the two S.S.F. windows were not detected in the trench profile at all. These windows would have been easily differentiated from other materials in that fluids provide no point resistance or local friction. The complete absence of these windows elicited additional laboratory work to arrive at a plausible explanation.

Laboratory testing indicated that the soil/bentonite backfill does in fact exhibit significant strength in the undrained condition[22]. However, as drained conditions are approached over time, the backfill loses most of its cohesive strength. The drained cohesive strength of the backfill does not appear high enough to support a cavity in the wall via arching. It is therefore probable that after the cavity was created, it remained stable for a short time as supported by the relatively high undrained strength. With time, the backfill approached drained conditions with a resulting loss in strength. The top of the cavity then progressively failed until the original cavity became filled. The cavity thus propagated towards the surface of the trench. Evidence from sounding GZ-5 indicates that the failure zone may never have reached the surface. An anomalous zone was detected extending from 5 to 8 feet deep. In this zone, the friction ratio was very low for intact backfill. In fact, it corresponds to that indicative of a L.D.C.G window. However, no significant deviation in point resistance was obtained as would be the case for a L.D.C.G. window (See Section 3.3.3.3). This response may indicate the presence of a very coarse and very low density material as would be anticipated based on the revealing scenario.

In summary, laboratory testing of the strength properties of the granular soil/bentoinite backfill and field measurements taken during construction indicate that the support slurry filled windows were created during test section construction. However, they would be expected to be a transient phenomenon. It is therefore likely that they did not exist intact at the time the soundings were executed.

---

[22] Cohesive soils behave differently depending on the rate of load application as compared to the rate at which the pore pressure within the soil can change in response to the loading. The two extremes in this behavior are referred to as undrained (rapid loading) and drained (slow loading). Additional discussion can be found in Lambe 1969, pg. 423.

3.3.3 Test Section Soundings

Prior to sounding, the test section was allowed to stabilize for a period of three months after its construction. In this way, full consolidation of the backfill materials under their own weight was ensured. In addition, it was anticipated that creep effects were sufficiently accounted for via this time period[23]. As such, the test section should adequately model the conditions existing in the two year old Gilson Road containment wall.

Ten soundings were conducted in the test section spaced at approximately 2 foot intervals along the trench alignment (see Figure 3-11). The soundings were executed in two phases. The objective of the first phase, consisting of soundings GZ-1 to GZ-6, was to establish baseline data for the three standard piezocone indices; point resistance, local friction and excess pore pressure. The second phase objective was to investigate the behavior of the relative hydraulic conductivity index during the remaining four soundings; GZ-7 to GZ-10 inclusive. The data obtained for each sounding is included herein as Figures 3-12 through 3-21.

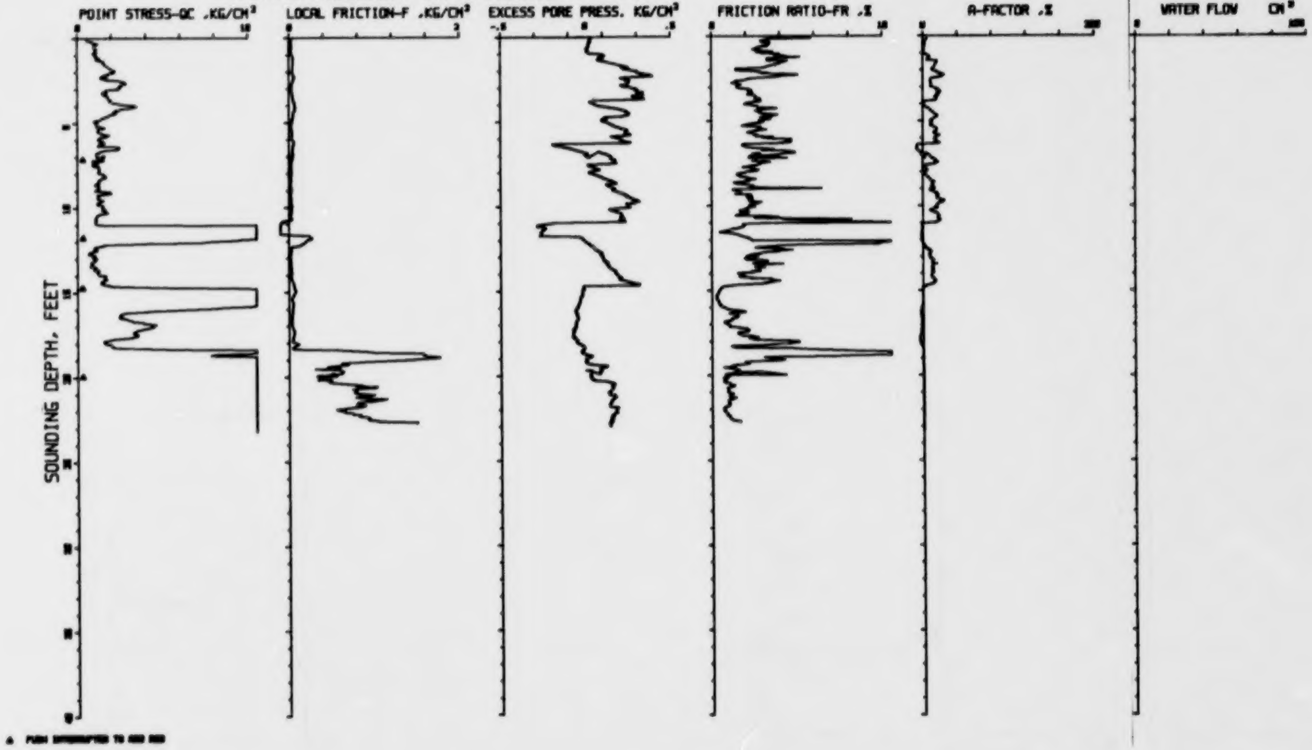
Before the index values could be established for the intact backfill and windows, the location of these materials within the test section needed to be established. Depth measurements alone were expected to provide sufficient information for this task as based on the construction profile (Figure 3-10). However, the discrepancies encountered between the actual as-built profile (sounding data) and the anticipated construction profile complicated this process. As discussed in Section 3.3.2, the test section did not provide the expected known condition pursuant to window location. Therefore, window locations were determined based on the sounding data and the expected qualitative piezocone response for the individual window materials. The piezocone response values were then quantified once window locations were established.

The following subsections are organized to first present data for piezocone response in the intact backfill followed by evaluation of window induced deviations from the backfill indices. The data from each sounding was correlated into a profile view of the test section which summarizes detected window locations (Figure 3-11). The variation between the window locations detected by the piezocone and the initial placement of the windows during test section construction was previously discussed in Section 3.3.2.

---

[23] Creep effects following consolidation typically follow an exponential decay function. See Lambe 1969, pg. 419.

FIG. 3-12 PIEZOCONE TEST--SOUNDING NO. GZ1



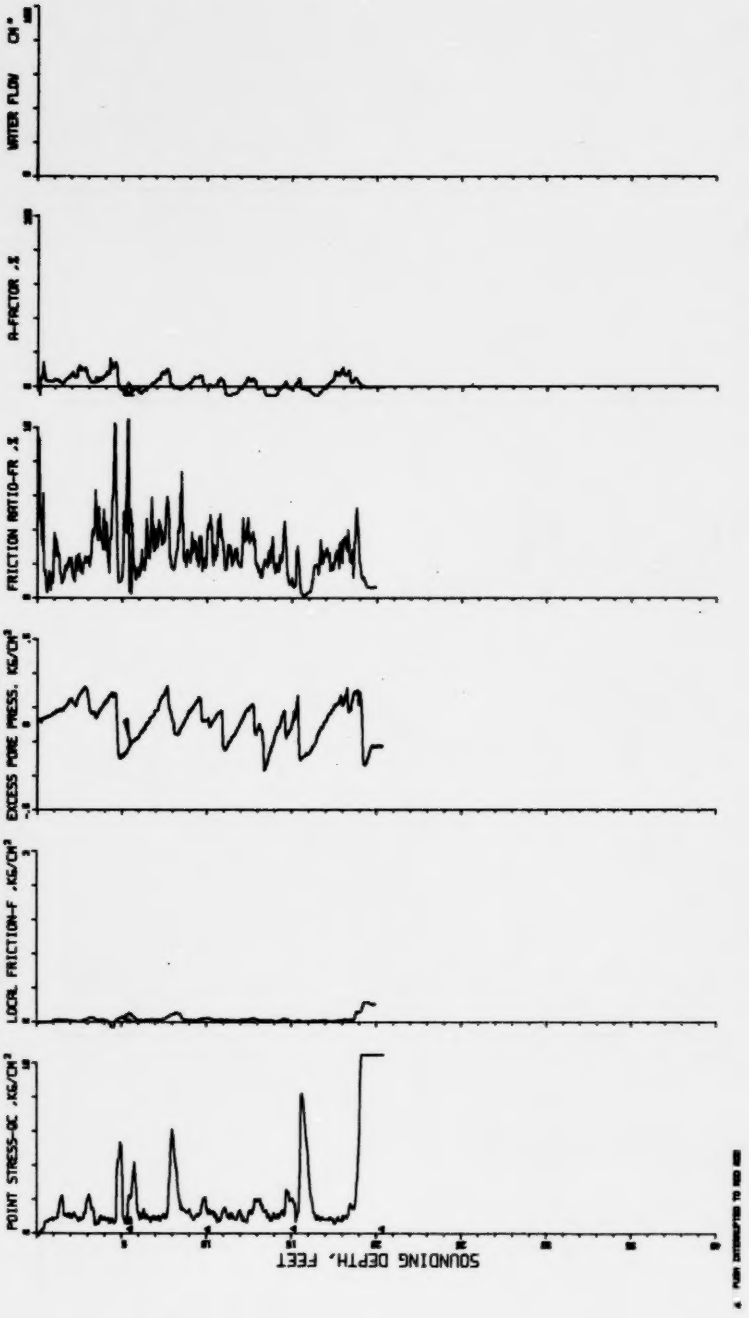


FIG.3-13 PIEZOCONE TEST--SOUNDING NO. GZ2

FIG. 3-14 PIEZOCONE TEST--SOUNDING NO. GZ3

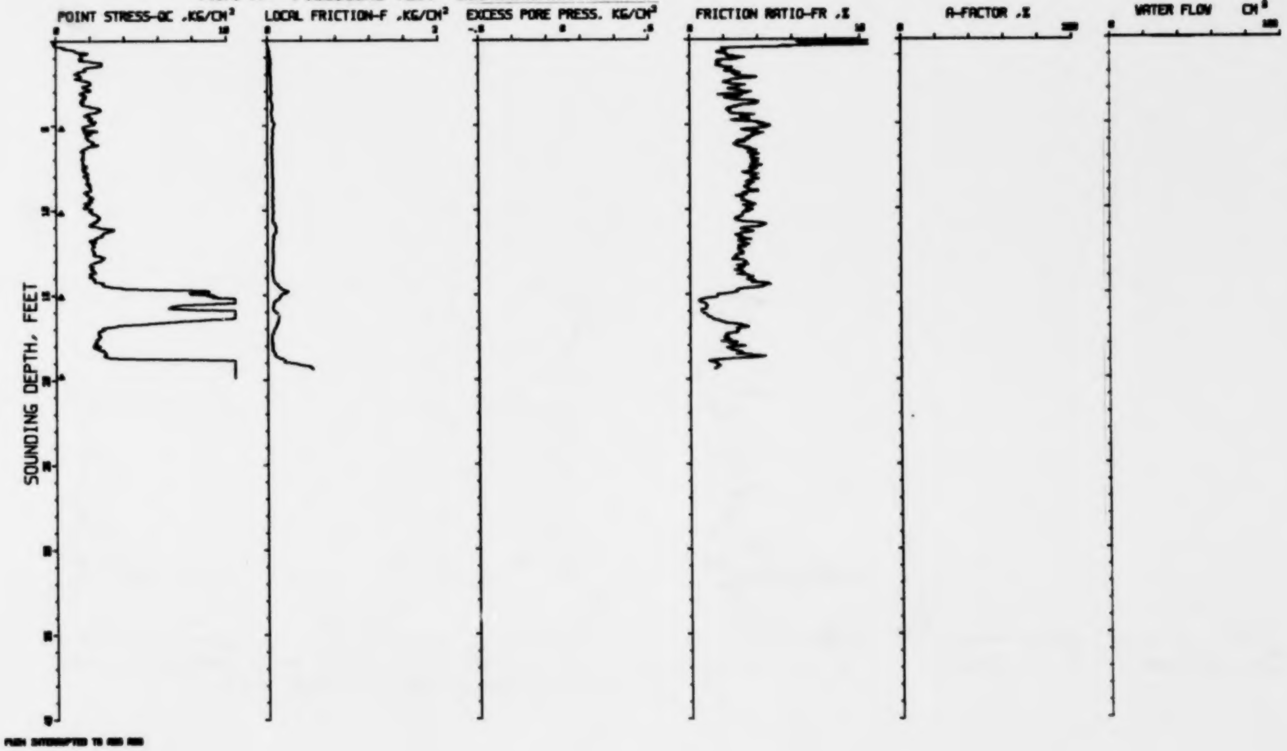


FIG. 3-15 PIEZOCONE TEST--SOUNDING NO. GZ4

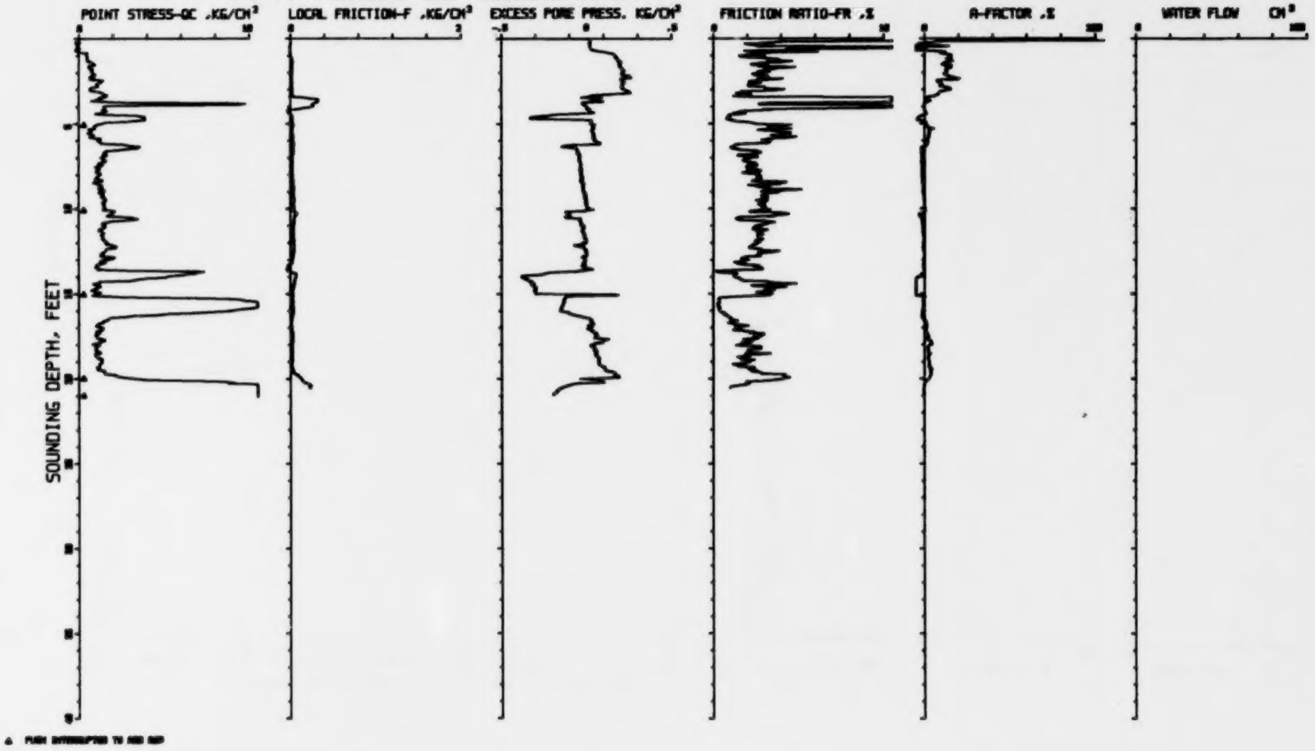
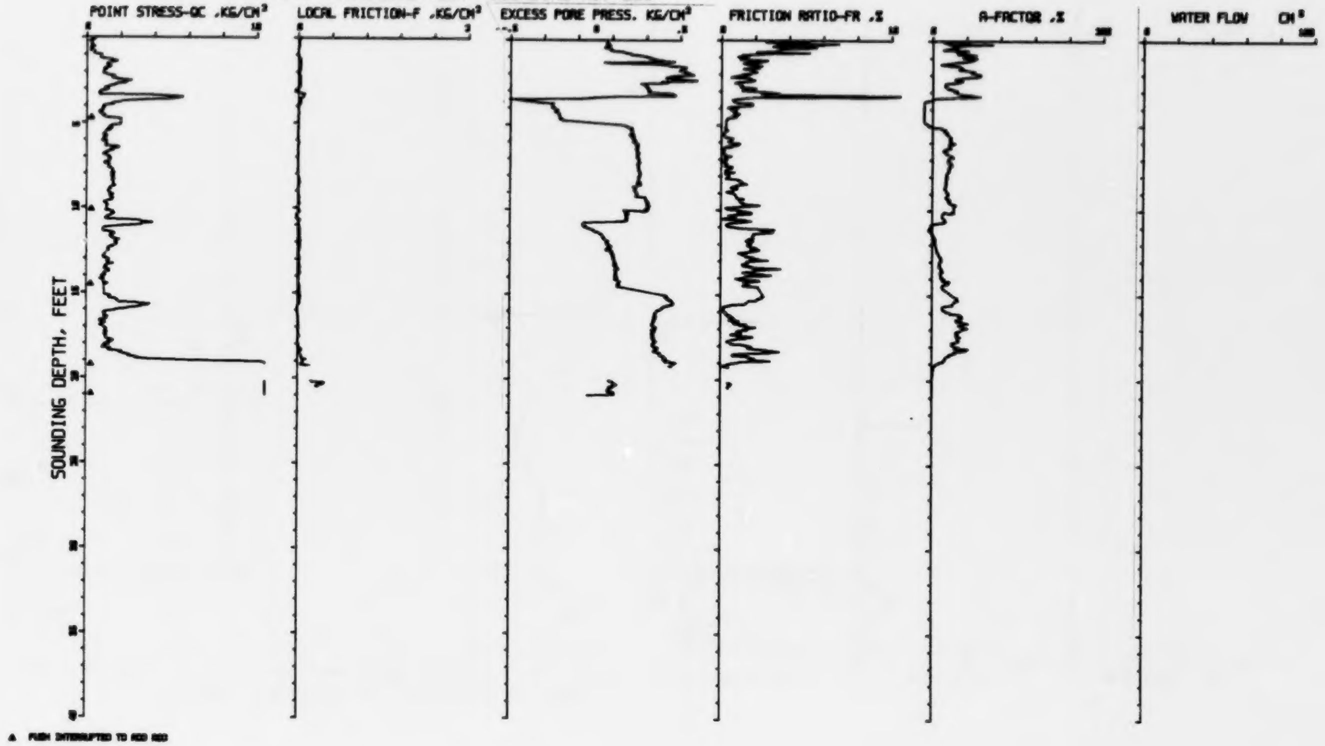


FIG.3-16 PIEZOCONE TEST--SOUNDING NO. GZ5



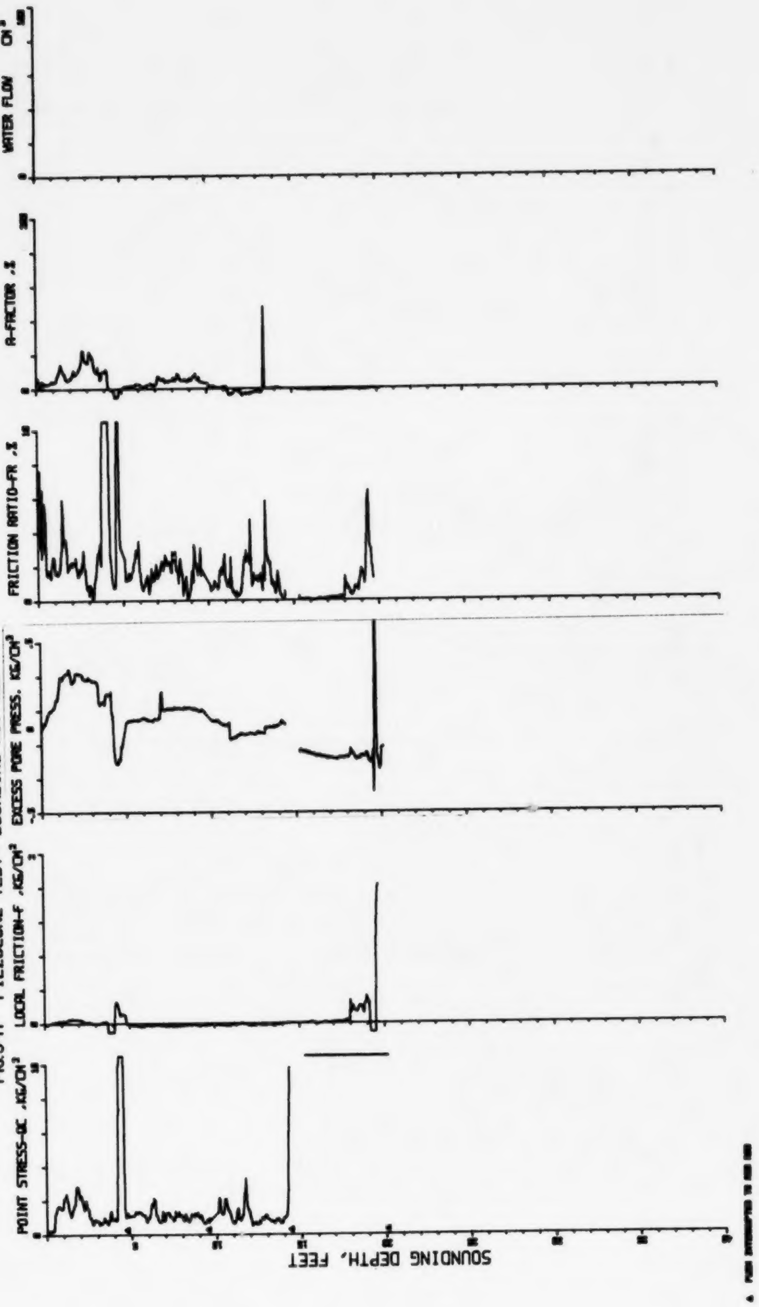


FIG.3-17 PIEZOCONE TEST--SOUNDING NO. GZ6

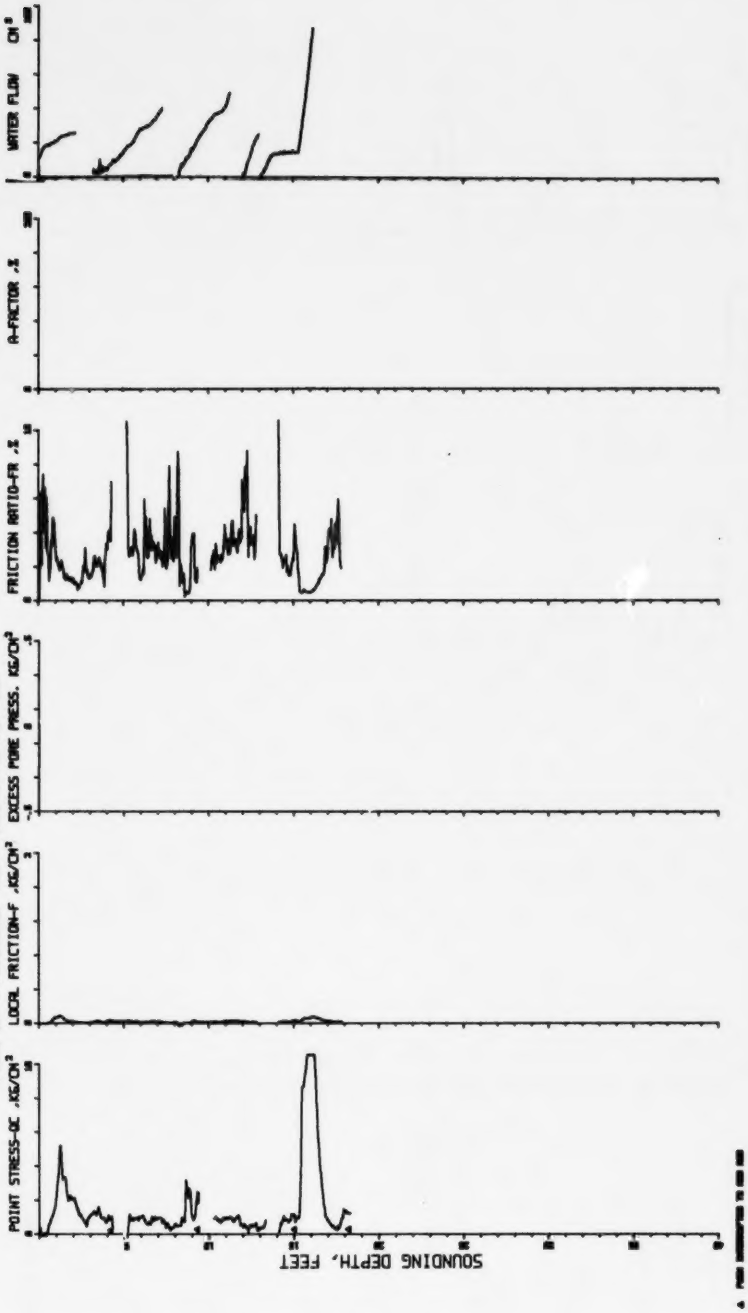
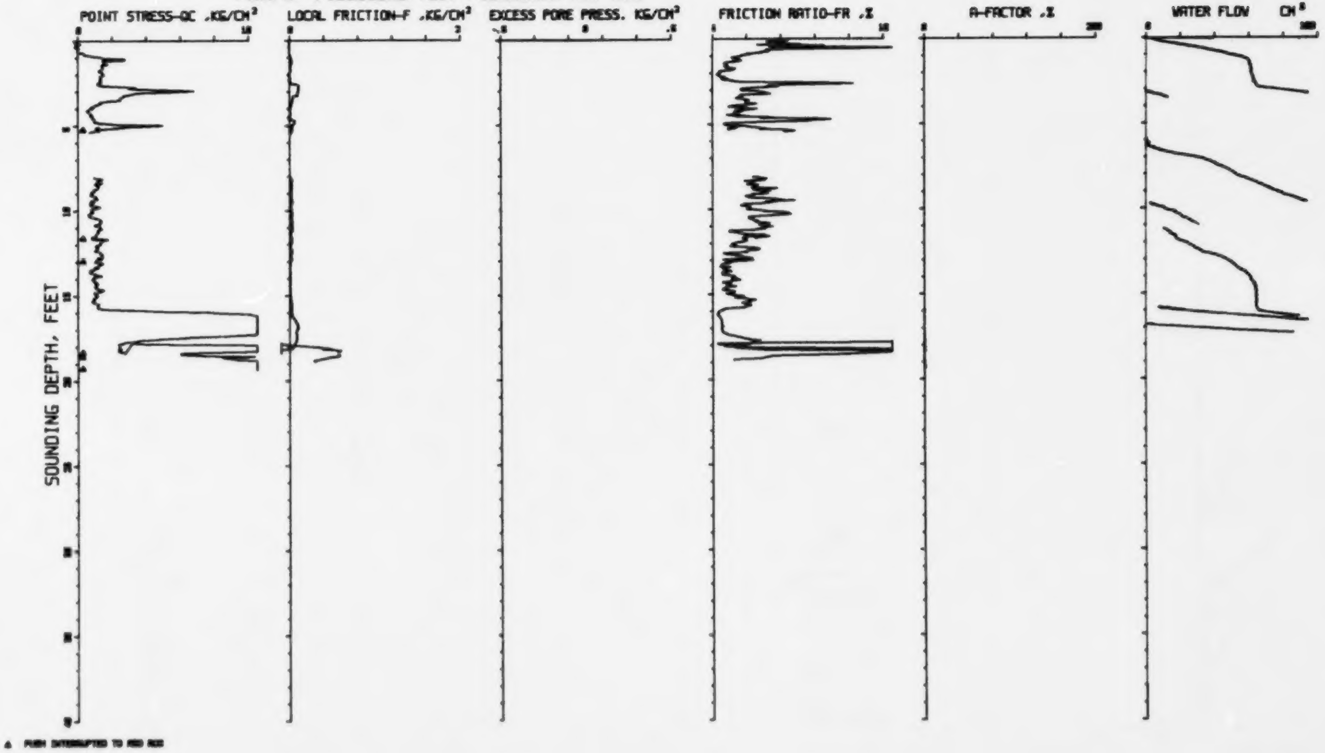


FIG.3-18 PIEZOCONE TEST--SOUNDING NO. GZ7

FIG.3-19 PIEZOCONE TEST--SOUNDING NO. GZ8



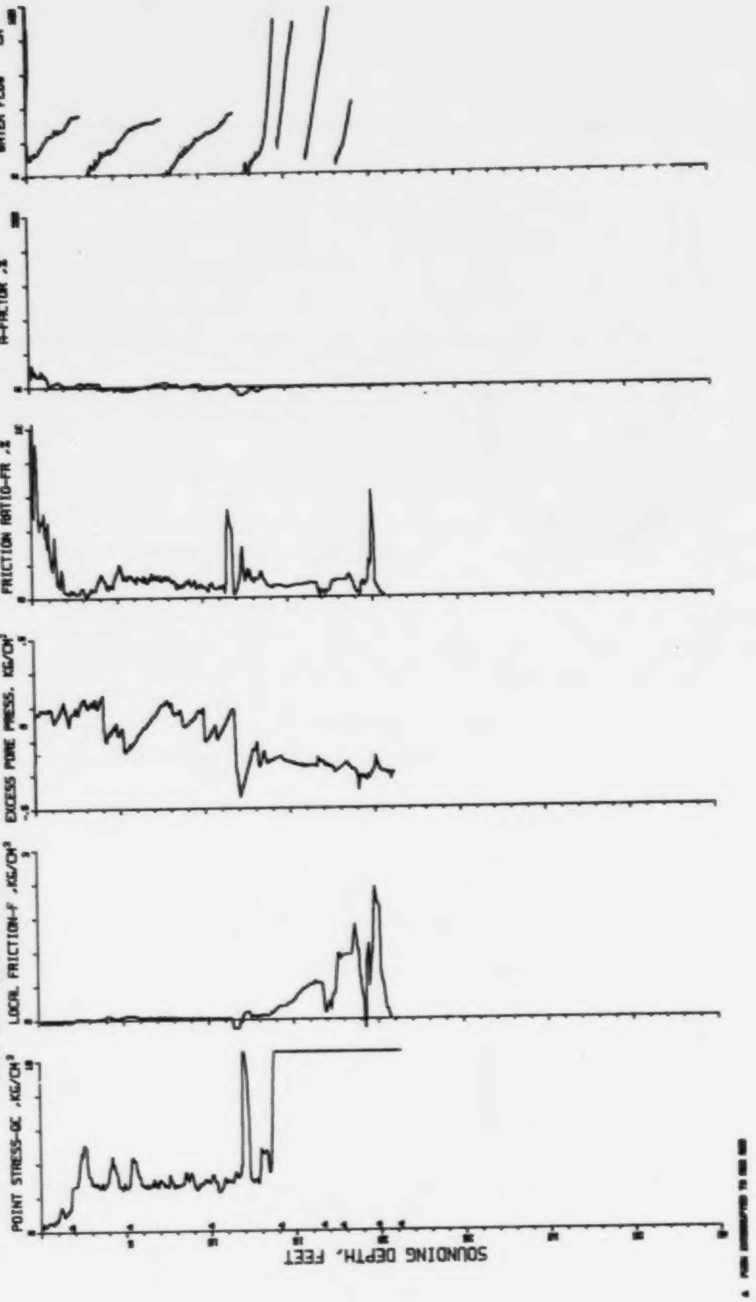
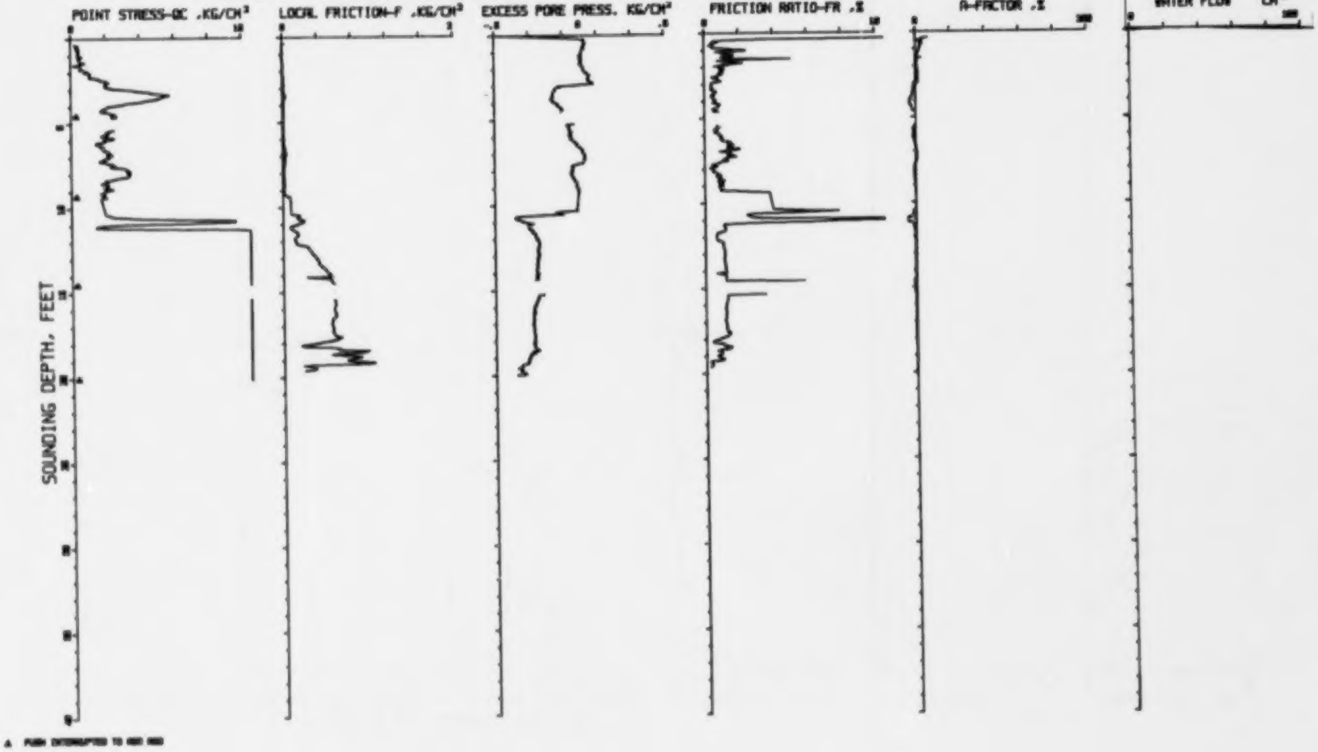


FIG.3-20 PIEZOCONE TEST--SOUNDING NO. GZ9

FIG.3-21 PIEZOCONE TEST--SOUNDING NO. GZ10



3.3.3.1 Intact Backfill Analysis

The first step in analysis of the sounding data encompassed identification of zones of intact backfill. Zones of intact backfill were identified via visual inspection of the sounding data as based on the following guidelines:

- Depth of the sounding as compared to the anticipated placement of materials in the test section (Figure 3-10).
- Consistency of the data with trends expected based on theoretical considerations (Section 3.2.6.1), and
- General uniformity of the data over a given zone.

The locations of intact backfill zones are summarized in Figure 3-11. Piezocone response in the backfill was then evaluated and average response values for the individual indices parameters of point resistance, local friction, excess pore pressure, and relative hydraulic conductivity were established.

Average response values were determined via linear interpolation performed directly on the graphs. These data are summarized in Tables 3-3 and 3-4. Deviations from these values formed the basis for subsequent window identification.

TABLE 3-3

INTACT BACKFILL

Sounding Number	Average Point Resistance (kg/cm^2)	Average Frictional Ratio (%)	Excess Pore Pressure (kg/cm^2)	Notes
GZ-1	1.3	2.6	+0.2	
GZ-2	1.2	2.8	0	
GZ-3	2.0	3.1	--	
GZ-4	1.3	2.9	0	
GZ-5	1.3	1.8	+0.2	1
GZ-6	1.2	1.5	+0.1	
GZ-7	1.0	2.8	--	
GZ-8	1.3	2.0	--	
GZ-9	2.8	1.1	0	
GZ-10	2.1	0.9	0	
Average	1.5	2.2	+0.1	

Notes:
1. Abnormally low friction ratio (0.3) encountered above zone of attempted support slurry window. This value omitted in average for GZ-5.
2. Highest and lowest values omitted from average.
3. -- indicates data not taken.

TABLE 3-4

RELATIVE HYDRAULIC CONDUCTIVITY INDEX

Material Type	Sounding Number	Depth (ft.)	Average Water Take[1] cm^3/ft	Notes
Intact	GZ-7	0.5- 2.3	4.9	
Intact	GZ-7	4.0- 7.4	10.1	
Intact	GZ-7	8.4-11.4	15.9	
Intact	GZ-7	12.0-13.0	29.4	
Intact	GZ-8	1.3- 2.8	2.9	
Intact	GZ-8	7.1- 9.7	23.3	
Intact	GZ-8	9.8-11.0	20.4	
Intact	GZ-8	11.2-14.0	18.9	
Intact	GZ-9	0.0- 3.0	8.2	
Intact	GZ-9	3.6- 6.2	9.0	
Intact	GZ-9	6.2- 7.9	2.6	
Intact	GZ-9	8.0-12.0	8.4	
Intact	GZ-9	12.8-13.8	11.8	
Weighted Average			12.2	
H.D.C.G.	GZ-9	13.8-14.7	111.1	
H.D.C.G.	GZ-9	14.9-15.8	71.4	
H.D.C.G.	GZ-9	16.4-17.8	62.5	
H.D.C.G.	GZ-9	18.4-19.1	52.6	
Weighted Average			74.0	
L.D.C.G.	GZ-7	15.4-16.4	90.9	
L.D.C.G.	GZ-8	15.8-15.9	166.7	
L.D.C.G.	GZ-8	15.9-16.8	125.0	
L.D.C.G.	GZ-8	17.0-17.6	166.7	
Weighted Average			123.1	
S.F.G.	GZ-8	2.7- 3.3	71.4	
Weighted Average			71.4	

Notes:

1. Defined as the volume of flow into the formation per unit depth penetrated.
2. Weighted average computed based on material thickness.
3. 2.5 psi driving pressure (above hydrostatic) used for testing.
4. H.D.C.G. - High Density Clean Granular Window
5. L.D.C.G. - Low Density Clean Granular Window
6. S.F.G. - Slurry Filled Granular Window

A. Point Resistance

In general, point resistance in the intact backfill was nearly constant with depth for a given sounding. The only exception was sounding GZ-3 (Figure 3-14) where point resistance increased somewhat with depth.

The magnitude of average point resistance from sounding to sounding ranged from 1.0 kg/cm^2 to 2.8 kg/cm^2. When the individual averages for each sounding were again averaged, the value obtained for point resistance in the backfill was 1.5 kg/cm2. This value fell below the range of point resistance given on empirical correlations (see Figures 3-5 and 3-6). As such, the very low point resistance indicated that the backfill was very soft.

B. Local Friction

The values of local friction measured in the intact backfill were very low, typically less than 0.05 kg/cm^2. As can be seen on the figures, the standard software scaling factors could not provide adequate resolution for measurements in the intact backfill while still remaining within scale for the entire sounding. However, the graph of friction ratio did provide adequate resolution.

The magnitude of average friction ratio for each of the ten soundings ranged from 0.9 percent to 3.1 percent. The average friction ratio for the ten soundings combined was 2.2 percent. The very low value of point resistance (1.5 kg/cm^2) combined with the friction ratio indicated that, although the backfill was primarily granular (by dry weight), it behaved as a plastic material due to the bentonite. This is demonstrated on Figure 3-6 which indicates that the backfill is a very soft clayey sand/silt.

C. Excess Pore Pressure

The excess pore pressures generated while sounding the backfill were small in comparison to the operational range of the transducer incorporated in the piezocone (7 kg/cm^2). In particular, the data magnitude was small when compared to possible errors in establishing the zero point for the transducer. As such, the data was interpreted in a qualitative sense only.

The soundings yielded excess pore pressures that averaged near zero or slightly positive (up to +0.2 kg/cm^2). Spikes in the data ranged from a +0.6 to a -0.2 kg/cm^2. Based on theoretical considerations, it is unlikely that the intact backfill should yield negative excess pore pressures (see Section 3.2.6.1). It is therefore probable that either the value used for hydrostatic pore pressure[24] was too high or the transducer zero stability was not adequate due to its high range. However, if an average excess pore pressure of +0.1 to +0.2 kg/cm^2 is used for the backfill, then the data indicate that the material is a clayey sand (see Figure 3-7).

D. Relative Hydraulic Conductivity

The permeameter equipment was employed to obtain water take values in the backfill as an index of relative permeability. This equipment was used for soundings GZ-7 through GZ-10, only. The graph of water take vs. depth for each sounding was divided into regions where the intact backfill exhibited a uniform rate of flow. In all cases, data from the upper 12 inches of the test section soundings was disregarded. This procedure was followed in recognition of the preferential boundary flow conditions which may have existed between the piezocone drill string and the backfill[25]. The water take value for each zone was then established by visual linearization of the curve. This data is summarized on Table 3-4. The water take values ranged from 2.6 cm^3/ft to 29.4 cm^3/ft. A weighted average (based on zone thickness) of 12.2 cm^3/ft was computed when all the intact backfill was considered.

---

[24] Hydrostatic pore pressure is subtracted from the total pore pressure measured by the transducer to compute excess pore pressure.

[25] Upon insertion, it is likely that a preferential flow channel would form along the drill string due to boundary effects. Higher water takes would therefore be expected due to the communication of this flow channel with the surface. This effect can be seen in soundings GZ-7 and GZ-8. Based on this data, a 12 inch zone was established where the values obtained were considered inaccurate.

3.3.3.2 High Density Clean Granular Window

The high density granular window (H.D.C.G.) was simulated by the unexcavated sand and gravel formation remaining below the trench bottom. The data (summarized in Tables 3-4 and 3-5) indicatedthat point resistance and relative hydraulic conductivity were the most useful parameters for detecting this type of window. The high value of local friction and the low friction ratio also allowed discrimination between the window and the backfill. The response was less abrupt, however. This was probably due to the 20 cm length of the friction sleeve. Excess pore pressure, although qualitatively useful as corroboratory data, provided little quantitative identification. It is expected that a more sensitive transducer (lower operational range) would provide better performance. The locations of H.D.C.G. windows within the test section are summarized in Figure 3-11. The following subsections treat these parameters in more detail on an individual basis.

TABLE 3-5

HIGH DENSITY CLEAN GRANULAR WINDOW

Sounding Number	Depth (Ft.)	Point Resistance[5] kg/cm^2	Frictional Ratio (%)	Excess Pore Pressure (kg/cm^2)	Water Take	Notes
GZ-1	19.0-23.2	100	1.0	+0.1	--	
GZ-2	19.0-20.5	45	0.6	-0.3	--	
GZ-3	18.8-20.0	65	1.0	-	--	
GZ-4	20.0-21.0	65	1.0	-0.2	--	
GZ-5	19.0-21.0	90	0.6	-0.1	--	
GZ-6	19.6-20.1	100	--	-0.3	--	
GZ-7	--	--	--	-	--	1,2,3
GZ-8	18.8-19.4	40	--	-	--	1,2
GZ-9	13.8-21.2	90	0.7	-0.3	Yes	
GZ-10	11.3-20.0	100	1.0	-0.2	--	
Average[4]		69	0.9	-0.1 to -0.2		

Notes:
1. Friction sleeve did not penetrate window.
2. Permeameter did not penetrate window.
3. Sounding stopped before window.
4. Average computed with omission of highest and lowest values.
5. Point resistance greater than 10 kg/cm^2 is off-scale on figures. Peak data taken from digital output.
6. -- indicates data not taken.

A. Point Resistance

Point resistance provided a clear indicator when this window was encountered. The values of point resistance ranged from 40 kg/cm^2 to over 100 kg/cm^2. The lower end of this range is considered artificially reduced in that penetration was typically terminated before full point resistance was reached so as to protect the transducer from an overload condition. The average value of point resistance for the H.D.C.G. window was 69 kg/cm^2 as compared to 1.5 kg/cm^2 for the intact backfill.

B. Local Friction

Local friction values for the H.D.C.G. window can only be established in cases where the entire friction sleeve penetrated the window. This required a point penetration distance of nearly 1 foot. Therefore, soundings GZ-6, GZ-7, and GZ-8 did not provide local friction data. The average value of local friction obtained from the remaining soundings (0.44 kg/cm^2) was significantly above that yielded by the intact backfill (<0.05 kg/cm^2).

Friction ratio values were also computed for the H.D.C.G. window. They ranged from 0.6 percent to 1.0 percent with an average 0.9 percent. This value is less than one-half that exhibited by the intact backfill (average = 2.2 percent) and, along with point resistance, is indicative of a dense, coarse grained noncohesive material (Figure 3-6). This classification,based on piezocone data, agrees well with the visual identification of the material.

C. Excess Pore Pressure

The magnitude of excess pore pressure generated when an H.D.C.G. window was encountered was small as compared to the transducer range. In general, however, they were negative as would be expected for a dense noncohesive material. The sounding data indicated that a reasonable average value was -0.1 to -0.2 kg/cm^2. This was best exemplified by soundings GZ-9 and GZ-10 where over 7 feet of the window were penetrated. This value was further corroborated by the other soundings with lesser penetration distances. However, GZ-1, which penetrated over 4 feet into the window, yielded positive excess pore pressures. This anomalous behavior could not be explained based on the available data. A value of -0.1 to -0.2 kg/cm^2 for excess pore pressure and a point resistance of 69 kg/cm^2, indicated that the material is noncohesive as based on Figure 3-7. Again this matches the visual classification.

From the standpoint of window detection, it is also useful to examine at the change in excess pore pressure as compared to that for intact backfill. While the average value for the intact backfill (+0.1 kg/cm^2) was not greatly different from the average window generated value (-0.1 to -0.2 kg/cm^2), the differences for each sounding were more significant. The typical response was a significant decrease in excess pore pressure at the interface between the backfill and the window. On average, this change amounted to -0.4 to -0.5 kg/cm^2 in the cases where the window was located immediately below the backfill. Other cases were less definitive due to intervening windows of different types.

D. Relative Hydraulic Conductivity

The permeameter stone penetrated the H.D.C.G. window in only one sounding. Penetration into the window required over 2 feet of point penetration due to permeameter location on the instrument. In general, significant point penetration of the window was not possible due to limited point resistance transducer capacity.

As shown on sounding GZ-9, the hydraulic conductivity index increased significantly upon encountering the window. Four separate determinations of water take were made. As summarized on Table 3-4, the average value was 74.0 cm^3/ft. This is significantly above the average value for intact backfill of 12.2 cm^3/ft.

3.3.3.3 Low Density Clean Granular Window

As for the H.D.C.G. window, point resistance and relative hydraulic conductivity were the most dramatic parameters with respect to change when encountering a low density clean granular window. However, point resistance was not as definitive as for the H.D.C.G. window due to the lower density and lack of gravel size constituents. While change in friction ratio provided good correlation with point resistance determined window location, local friction values were too low for discrimination on the graphs. Excess pore pressure was useful only in a qualitative sense, as was the case for the H.D.C.G. window.

The data for the above parameters are summarized in Tables 3-4 and 3-6. The locations of the L.D.C.G. windows are shown in Figure 3-11. The following subsections discuss each parameter in more detail.

TABLE 3-6

LOW DENSITY CLEAN GRANULAR WINDOW

Sounding Number	Depth (Ft.)	Point Resistance (kg/cm^2)	Friction Ratio (%)	Excess Pore Pressure (kg/cm^2)	Water Take	Notes
GZ-1	14.7-16.2	16	0.4	-.1	--	
GZ-1	16.2-17.8	4	1.0	-.1	--	1
GZ-2	15.3-16.2	8	0.2	-.2	--	
GZ-3	14.5-16.8	17	0.7	-	--	
GZ-4	4.4- 5.0	4	0.9	-.3	--	1
GZ-4	13.6-14.3	7	0.4	-.4	--	
GZ-4	15.2-16.5	11	0.3	-.2	--	
GZ-5	10.7-11.2	4	0.4	-.1	--	1
GZ-5	15.5-16.1	4	<0.2	+.4	--	1
GZ-6	14.3-18.5	16	<0.2	-.2	--	
GZ-7	1.0- 1.5	5	0.9	--	No	1,3
GZ-7	15.4-16.8	16	0.4	--	Yes	
GZ-8	15.8-17.7	19	0.5	--	Yes	
GZ-9	2.3- 2.8	5	0.4	--	No	1,3
GZ-9	4.0- 4.5	4	1.2	--	No	1,2,3
GZ-9	5.2- 5.8	4	1.2	--	No	1,2,3
GZ-9	12.9-13.4	4	1.1	--	--	1,2,3
GZ-10	3.0- 4.0	6	0.6	--	--	1,2
Average		10.3	0.4	0. to -0.2		

Notes:
1. Potential window - does not meet point resistance criteria, but point resistance significantly above intact backfill.
2. Window ruled out based on friction ratio criteria.
3. Window ruled out based on water take criteria.
4. Average computed with omission of highest and lowest values. Windows ruled out based on notes 2) and 3) above also omitted.
5. -- indicates data not taken.

A. Point Resistance

Due to the lower density and lack of gravel sizes in this window, point resistance was not expected to be as high as for the H.D.C.G. window. Guidelines were therefore established to discriminate between a potential window and the intact backfill as based on point resistance alone. Final determination of window existence was then based on confirmatory data from the other parameters.

Point resistance magnitudes greater than 7 kg/cm^2 were considered indicative of L.D.C.G. window material. This boundary was established, via inspection of the sounding data, at five times the magnitude previously established for intact backfill. Values of point resistance which were less than 7 kg/cm^2, but were still significantly greater than the response in the backfill, were also considered subject to confirmatory data from the other piezocone parameters.

As shown in Table 3-6, the average point resistance was 10.3 kg/cm^2. This value incorporated data from windows with point resistances less than the cutoff point of 7 kg/cm^2, but which could not be ruled out based on other piezocone parameters. This average is thus considered a lower bound average but was used in recognition that the 7 kg/cm^2 cutoff point was somewhat arbitrary. The lower bound average of 10.3 kg/cm^2 is still significantly above that generated by the intact backfill. The abrupt development of these relatively high point resistances, as compared to intact backfill, provided significant confidence that the L.D.C.G. windows locations could be identified.

B. Local Friction

The values of local friction measured in the L.D.C.G. windows were quite low, typically less than 0.1 kg/cm^2. At this magnitude of response, the difference between the backfill and the window could not be resolved on the graphs. However, the friction ratio provided a better index of window presence. Average friction ratio for the L.D.C.G. window was 0.4 percent as compared to 2.2 percent for the intact backfill. A friction ratio upper limit of 1.1 percent (one-half the average intact value) was established as a guideline for L.D.C.G. windowidentification.

The average value of friction ratio, for the L.D.C.G. window combined with an average point resistance of 10.3 kg/cm^2, indicated that the material was a loose noncohesive soil type(Figures 3-5 and 3-6). This description, from the empirical correlations, agrees well with the actual material type placed as the windows.

C. Excess Pore Pressure

The excess pore pressures generated while penetrating the L.D.C.G. window materials were typically slightly negative; average magnitudes ranged between -0.1 to -0.2 kg/cm^2 as was the case for the H.D.C.G. window. It would not be expected that sands existing at low relative densities would produce negative excess pore pressures. In fact, very loose sands, such as would be probable for the L.D.C.G. window (deposited in a submerged condition), would be expected to generate slightly positive or zero excess pore pressures. The low magnitude of the data, as compared to the transducer operating range, may have introduced a zero shift error. This hypothesis would result in zero excess pore pressure which would be more consistent with theoretical considerations and past empirical data.

Independent of the actual magnitude of excess pore pressure generated, the change in this parameter at the transition from the backfill to a L.D.C.G. window provided useful window identification data. In all cases, the trend was for an abrupt decrease in excess pore pressure when entering the window as would be expected. The average decrease was 0.3 kg/cm^2.

The actual values of negative excess pore pressure generated during the soundings indicated that the material fell outside the range of the empirical material identification chart (Figure 3-7). However, if the excess pore pressures are adjusted slightly in the positive direction, to account for the hypothesized zero shift, then the empirical correlation correctly identifies the sandy composition of the window. Even without adjustment, the correlation indicates that the material is likely to be noncohesive.

D. Relative Hydraulic Conductivity

The permeameter was operational during penetration of two windows satisfying the point resistance and friction ratio guidelines previously established to identify L.D.C.G. material. In both cases, the relative hydraulic conductivity index increased markedly. Four separate determinations of water take were made in those two windows as summarized on Table 3-4. The water take values ranged from 90.9 to 166.7 cm^3/ft. with a weighted average of 123.1 cm^3/ft.

The average value of relative hydraulic conductivity obtained for the L.D.C.G. windows were an order of magnitude greater than that representing the backfill (12.2 cm^3/ft.). As such, identification of these more permeable zones was straight forward. The relative hydraulic conductivity index was therefore also used to rule out marginal spikes in point resistance from further consideration as potential windows.

3.3.3.4 Slurry Filled Granular Window

The slurry filled granular windows were most easily identified based on the combination of high point resistance and high local friction, as compared to the intact backfill. The relative hydraulic conductivity index increased markedly in the S.F.G. window, but only one determination of its magnitude was made. Excess pore pressures typically exhibited negative trends but the magnitudes were low compared to the operating range of the transducer. Excess pore pressure was thus used as a qualitative index only.

The individual sounding data for the S.F.G. windows is summarized on Table 3-7. The locations of the windows are presented on Figure 3-11. The response of the individual piezocone indices is described in further detail in the following sections.

TABLE 3-7

SLURRY FILLED GRANULAR WINDOW

Sounding Number	Depth (Ft.)	Point Resistance (kg/cm^2)	Friction Ratio (%)	Excess Pore Pressure (kg/cm^2)	Water Take	Notes
GZ-1	10.8-12.4	25	>10	-.3	--	
GZ-1	18.2-19.0	12	>10	0	--	3
GZ-2	4.3- 5.3	5	>10	-.2	--	1
GZ-2	5.7- 6.0	4	1.5	-.1	--	1,2
GZ-2	7.5- 8.7	6	7	-.1	--	1
GZ-4	3.4- 4.0	10	>10	-.1	--	
GZ-5	3.4- 3.9	5	>10	-.5	--	1
GZ-6	3.7- 4.8	17	>10	-.2	--	
GZ-6	18.5-19.5	18	6	-.2	--	3
GZ-8	2.7- 3.6	7	8	--	Yes	
GZ-8	4.7- 5.2	5	7	--	--	1
GZ-8	17.9-18.6	30	>10	--	--	3
GZ-9	11.3-12.5	10	5	-.4	--	
GZ-10	10.2-11.1	10	9	-.3	--	3
Average[4]		11.4	7.9	-0.2		

Notes:
1. Potential window - does not meet point resistance criteria.
2. Window ruled out based on friction ratio criteria.
3. Filter cake and/or formation soil mixed with slurry by bucket teeth at trench bottom.
4. Average computed with omission of highest and lowest values. Windows ruled out based on Note 2) above also omitted.
5. -- indicates data not taken.

A. Point Resistance

The point resistance encountered when penetrating the S.F.G. window was similar to that for the L.D.C.G. window. The average value computed was 11.4 kg/cm^2 as compared to 10.3 kg/cm^2 for the L.D.C.G. window. The range of point resistance obtained from the individual windows was also comparable. This result was expected based on the similar composition and depositional methods used to construct the windows. The same point resistance guidelines established to identify L.D.C.G. windows were therefore also adopted for this window type.

B. Local Friction

The magnitude of local friction generated when penetrating the S.F.G. window was often greater than 0.1 kg/cm^2 with an average value of 0.2 kg/cm^2. This value was high as compared to that for intact backfill and the L.D.C.G. window. The high average local friction for the S.F.G. windows did not produce friction ratios significantly greater than the intact backfill at the point of maximum point resistance. In fact, the average point resistance and average local friction values combine to yield an average friction ratio of 1.8 percent. This is identical to that for the intact backfill (2.2 percent) to within the accuracy of the data. However, high friction ratios were typically detected immediately above and below the peak in point resistance. This behavior was not evident for any of the other test section materials. It is hypothesized that the point resistance, when initially entering a S.F.G. window, was low due to lack of confinement by the soft, plastic backfill. This would also be true when the point approaches the bottom of the window[26]. The local friction, however, would be high over the entire window thickness due to the adhesion attributed to the bentonite filling the soil voids. Therefore, friction ratios would be high entering and exiting the S.F.G. windows.

A friction ratio of greater than 5 percent was required for high point resistance to be considered a S.F.G. window. The friction ratio reported for this window was the highest value recorded when the window was entered and exited. The thickness of the window summarized on Table 3-7 also reflected the occurrence of high friction ratio in the context of the above hypothesis.

The above data yielded an average friction ratio of 7.9 percent for the top and bottom of the window and an average value of 1.8 percent at the location of maximum point resistance. An average friction ratio of 7.9 percent, in conjunction with an average point resistance of 11.4 kg/cm^2, indicated that the window material is a mixed soil type as per Figure 3-6. The lower average friction ratio (1.8 kg/cm^2) yields a loose sand material identification. Both classifications are supported by the material type used to construct the window. The important point, however, is that a S.F.G. window yields a consistent piezocone signature and thus can be identified.

---

[26] This hypothesis is supported by the behavior of end bearing piles in multilayered systems (Poulos, 1980).

C. Excess Pore Pressure

Excess pore pressures generated when sounding a S.F.G. were similar to those representing the L.D.C.G. window, but somewhat more negative. The average value was -0.2 kg/cm^2. The change in excess pore pressure at the transition from the backfill to the window was also typically abrupt and in the negative direction. The average change was -0.5 kg/cm^2. Again, the magnitude of the change was greater for the S.F.G. window than the L.D.C.G. window.

General discussion of the excess pore pressure generation behavior in loose sand materials applies to the S.F.G. window as well as the L.D.C.G. window. This discussion is found in Section 3.3.3.3. An explanation for the greater negative response in the S.F.G. is not possible given the available data.

D. Relative Hydraulic Conductivity

The permeameter penetrated only one slurry filled granular window. Upon entering the window, from the intact backfill above, the water take abruptly increased from 2.9 cm^3/ft. to 74 cm^3/ft. As shown in the GZ-8 sounding data (Figure 3-19), the water take maintained a constant rate until the permeameter pressurization reservoir emptied and required refilling.

The water take obtained in the S.F.G. window was high as compared to the intact backfill as would be expected. However, it was also approximately 60 percent of the value obtained for the L.D.C.G. window. Given that both windows were constructed of the same soil material (concrete sand), the difference in water take can be attributed to two differing conditions. The first concerns pore fluid. The S.F.G. window contains bentonite slurry in the void spaces which is about 40 percent more viscous than clean water[27]. The second condition relates to window size. As previously indicated, the water take for a window totally entrapped in intact backfill will be related to the flux through the window/backfill interface (Section 3.2.6.3). The available data, as summarized on Figure 3-11, indicated that the L.D.C.G. window was probably significantly larger than the S.F.G. window. Both of the above conditions should result in a decreased relative hydraulic conductivity index for the S.F.G. window as compared to the L.D.C.G. window.

3.3.3.5 Support Slurry Filled Window

As discussed in Section 3.3.2.4, the support slurry filled window was not detected. It is hypothesized that the slurry filled cavity collapsed prior to the piezocone soundings.

3.3.4 Window Identification Guidelines

Based on the data obtained from the test section soundings, a set of general guidelines were developed to quantitatively model the signature corresponding to each type of window investigated. These guidelines were formulated as deviations in the piezocone indices obtained when sounding a window as compared to average values obtained in the intact backfill. These guidelines, summarized below by individual window type, were used to analyze the containment wall soundings for potential window locations.

It is emphasized that zones labeled as "potential windows" were identified based solely on deviations in piezocone indices as compared to values associated with intact backfill. Without confirmatory data, the existence of an actual window cannot be determined. Furthermore, the impact of the potential windows on overall containment wall effectiveness cannot be evaluated as based on piezocone data. Such evaluations can only be made by hydraulically stressing the cutoff wall and analyzing its effect on the hydrologic response of the aquifer/aquiclude system. This work is discussed in Section 4.

---

[27] As determined via marsh funnel testing. The support slurry used exhibited a marsh funnel of approximately 41 seconds, where as clean water has a marsh funnel viscosity of 29 seconds.

It should also be recognized that the test section upon which the following guidelines were based was constructed to model the type of backfill used at the Gilson Road site. The test section backfill was therefore a granular soil/bentonite type mix[28]. Other types of soil/bentonite mixes, or different backfill types all together (cement/bentonite for example), would not be expected to follow the same guidelines.

3.3.4.1 High Density Clean Granular Window

- Point resistance greater than twenty times that for the intact backfill.
- Point resistance consistent with the known material type, strength, and density of the in-situ native formation (values > 30 kg/cm^2 for the Gilson Road site).
- Friction ratio less than one-half the value obtained in the intact backfill.
- Reduction in excess pore pressure from values associated with the intact backfill (to near zero or negative).
- Relative hydraulic conductivity index greater than five times that for the intact backfill.

3.3.4.2 Low Density Clean Granular Window

- Point resistance greater than five times that for the intact backfill.
- Friction ratio less than one-half that corresponding to the intact backfill.
- Reduction in excess pore pressure to a near zero magnitude.
- Relative hydraulic conductivity index greater than five times that for the intact backfill.

---

[28] Soil/bentonite backfills can be separated into granular or clayey type mixes. Granular mixes are those composed primarily of noncohesive materials (by dry weight) which rely heavily on small percentages of bentonite to achieve low hydraulic conductivity. Clayey mixes contain high percentages of plastic materials which reduce the backfill hydraulic conductivity to low values prior to bentonite augmentation.

3.3.4.3 Slurry Filled Granular Window

- Point resistance greater than five times that for the intact backfill.
- Relatively high local friction values, greater than twice the intact backfill magnitude.
- Friction ratio greater than twice the intact backfill value upon entering and exiting the window.
- Reduction in excess pore pressure to a near zero magnitude.
- Relative hydraulic conductivity index greater than five times that for the intact backfill.

3.3.4.4 Support Slurry Filled Window

This type of window is not expected to remain stable in a granular type soil/bentonite backfill such as used at the Gilson Road site. However, this window type, if existing in a cutoff wall, would be easily identified through a lack of significant point resistance or local friction.

3.4 Containment Wall Soundings

Thirteen soundings were executed in the Gilson Road containment wall after completion of the test section trial soundings. Five of the soundings (GZ-11, GZ-12, GZ-13, S-2E, and S-10) were concentrated in two areas where windows were most likely to exist. The first area (Station 1+00) was the location of the deepest part of the containment wall. The second area (Station 3+45) coincided with the occurrence of a localized trench instability encountered during construction[29]. The remaining eight soundings were distributed along the downgradient portion of the containment wall. The individual locations of these soundings are provided in Figure 3-9.

The containment wall soundings were executed in two phases. The first phase (GZ-11, GZ-12, and GZ-13) followed immediately after the test section soundings. The second phase was delayed for approximately one month to allow modification of the permeameter equipment[30].

In general, the containment wall soundings exhibited trends similar to those corresponding to the intact backfill zones in the test section. Deviations in piezocone response from the intact backfill values were also noted in some cases. At depths where deviations were noted in one piezocone index, the other indices were examined to identify signatures resembling window data from the test section. The windows were analyzed based on the guidelines established during the test section soundings (Section 3.3.4).

An example sounding is analyzed in the following subsections. This sounding corresponded to the deepest portion of the cutoff wall (Section 1+00). This area was selected for investigation due to the 100 foot depth of the wall and thus the higher potential for windows. Of the three soundings performed in this area, GZ-13 exhibited the most complete record[31] and also provided the greatest variety of response. The record for sounding GZ-13 is provided as Figures 3-22a and 3-22b.

---

[29] During excavation, a 2 inch wide, 50 foot long, conchoidal shaped crack developed at the surface 15 to 20 feet behind the trench. It is hypothesized that this resulted from slurry loss into an esker deposit with the associated local instability at depth. Slurry was pumped to the trench from the two slurry ponds constructed in recognition of the esker deposits. The excess slurry head and overall stability was thus maintained. Thirty-six inch diameter pipes were then inserted into the trench to provide added support. The trench bottom was recleaned and the trench was backfilled.

[30] Initially, the permeameter porous stone was the same diameter as the drill string. The potential for preferencing boundary flow along the drill string was recognized during test section work. A step taper was therefore added to the design (Figure 3-2). In addition, it was found that a large permeameter pressurization/volume measurement reservoir was required. This equipment was also modified prior to the final phase of soundings.

3.4.1 Intact Backfill; Sounding GZ-13

The point resistance encountered in the soil/bentonite backfill exhibited a nearly constant trend at 2 kg/cm^2 to a depth of 50 feet. From 50 feet to the bottom of the sounding at 95 feet, a linearly increasing point resistance to 4 kg/cm^2 was evident. This data compared favorably with that developed during soundings in the test section (test section point resistance = 1.5 kg/cm^2). In fact, a trend of increasing point resistance with depth better follows the theoretical trend for a uniform material.

Local friction typically averaged 0.1 kg/cm^2 with a slightly increasing trend towards the bottom of the sounding. This value is somewhat higher than that established during the test section soundings (<0.05 kg/cm^2). The proportionally higher local friction, as compared to point resistance, also yielded a somewhat higher friction ratio of 3.0 to 3.5 percent (test section = 2.2 kg/cm^2). The point resistance and friction ratio values indicate that the material is a sandy/silty clay as based on the empirical identification charts (Figure 3-6).

Excess pore pressures generated during sounding GZ-13 were significantly higher than observed in the test section (0.2 kg/cm^2). The average magnitude ranged from 0.6 kg/cm^2 to 1.0 kg/cm^2, with the higher values encountered over the bottom section of the wall. These larger excess pore pressures are in better agreement with theoretical considerations than the data from the test section. This lends a higher degree of credibility to the hypothesis that the low, sometimes negative, average excess pore pressures may have been due to an inaccurate zero value. Figure 3-7 indicates that the material behaves as a clayey sand or a silty clay, as based on the excess pore pressure and point resistance indices.

Relative hydraulic conductivity values for the intact backfill averaged 7.6 $cm^3/ft.$, within a range of 4.5 $cm^3/ft.$ to 11.5 $cm^3/ft.$ With the exception of a few higher hydraulic conductivity zones, (potential windows) the water take values

---

[31] The excess pore pressure transducer failed during sounding S2E and the point resistance transducer failed in sounding S-10.

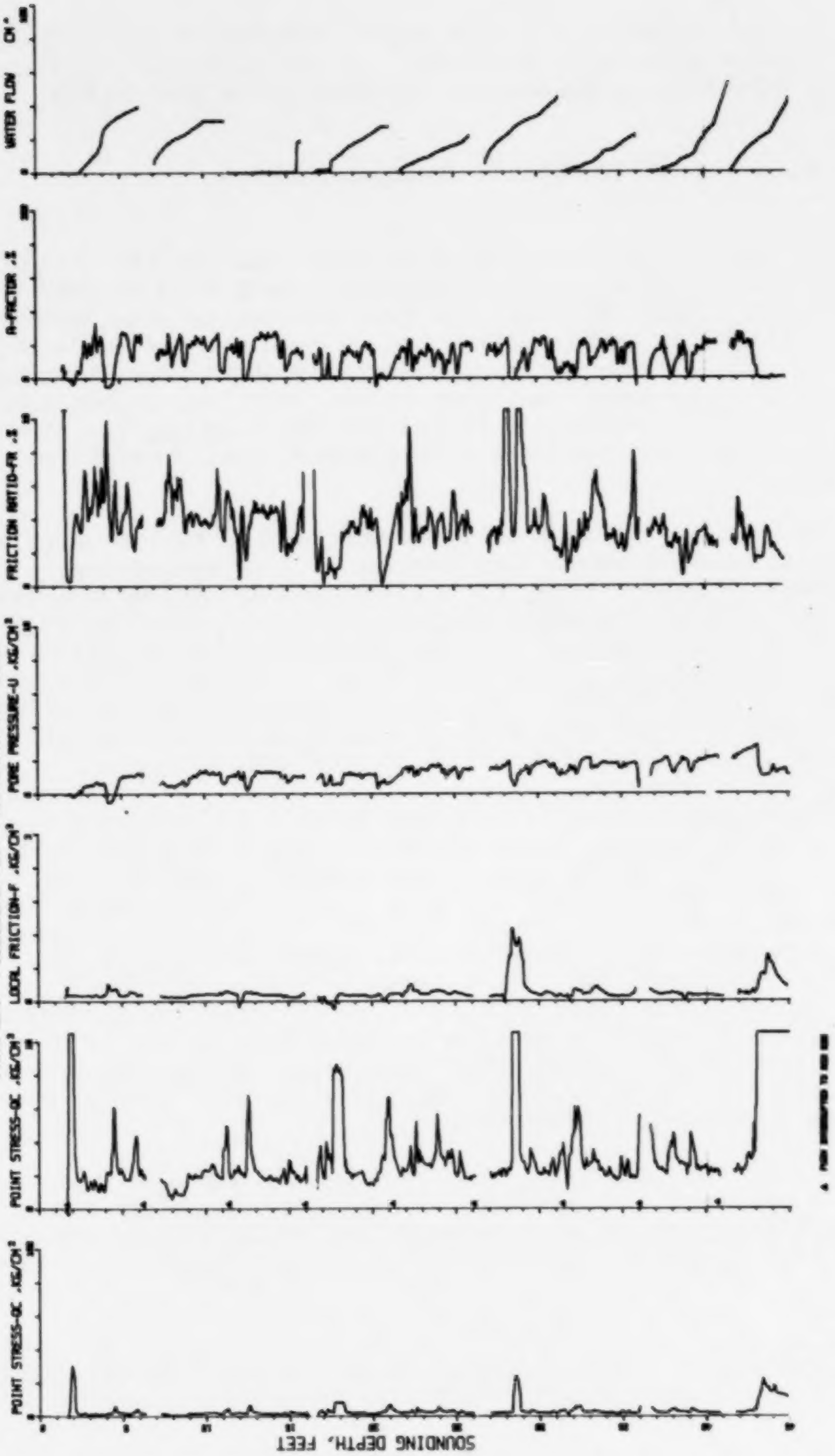


FIG. 3-22a PIEZOCONE TEST--SOUNDING NO. GZ13

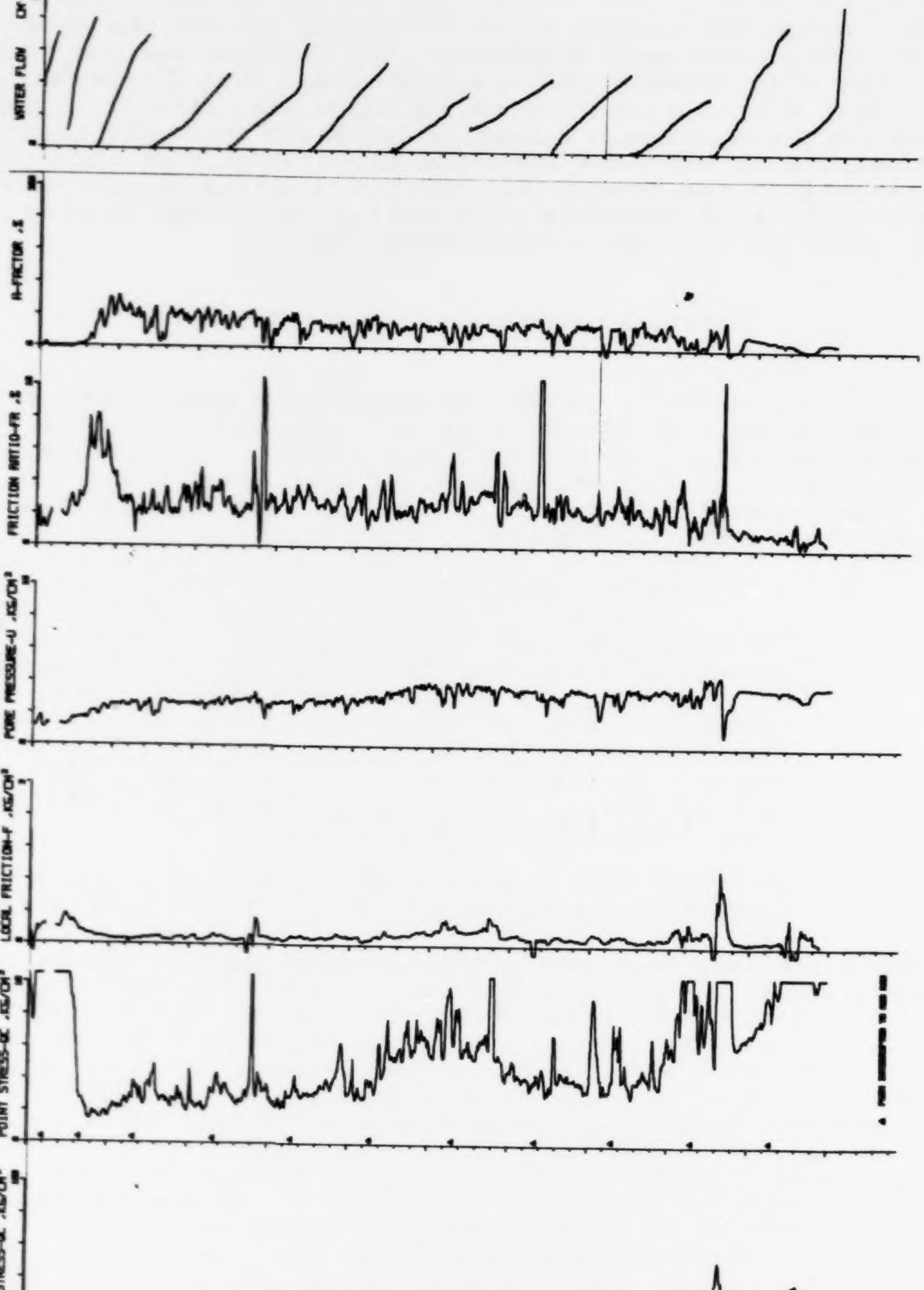


FIG.3-22b PIEZOCONE TEST--SOUNDING NO. GZ13

were relatively consistent. It should be noted that the head loss across the porous stone increased during the test section and containment wall soundings. The increase was due to smearing of the high density polyethylene stone and internal surface loading due to suspended solids (iron precipitate) in the water. Driving pressures were therefore increased after the test section soundings to maintain adequate flow rates. As such, the water take values are comparable within a given sounding but not generally from sounding to sounding (the same driving pressure was used for all test section soundings).

3.4.2 Potential Window Locations, GZ-13

Sounding GZ-13 exhibited signatures which corresponded to all three types of potential windows detected in the test section. These are the high density clean granular (H.D.C.G.) window, the low density clean granular (L.D.C.G.) window and the slurry filled granular (S.F.G.) window. Each window type will be discussed individually below.

A. High Density Clean Granular Window

The occurrence of this window type is most likely to be the result of incomplete excavation of native formation materials. This is particularly true of sites where the cutoff wall has been keyed to the top of bedrock as was the case at Gilson Road. Incomplete excavation to the top of bedrock typically results when the scale of bedrock surface convolutions is small relative to the size of the excavation tool[32].

As based on the previously established guidelines, it appears that sounding GZ-13 encountered native formation material at a depth of 88.5 feet. In this location the point resistance increased to over 50 kg/cm^2 and friction ratio decreased to about 1 percent (intact backfill values of 2 to 4 kg/cm^2 and 3.0 to 3.5 percent, respectively). Excess pore pressure dropped from an average value of +1.0 kg/cm^2 in the backfill to an instantaneous low of < -0.6 kg/cm^2 upon encountering this zone. The relative hydraulic conductivity index peaked in this zone at a maximum value of 40 cm^3/ft. (average value in intact backfill of 7.6 cm^3/ft.). With further penetration, the point resistance fell off dramatically (8 kg/cm^2) the relative hydraulic conductivity index decreased (7.4 cm^3/foot) and the excess pore pressure increased to positive values (+1 kg/cm^2). After three additional feet of penetration, this trend was again followed by high point resistance (40 kg/cm^2), markedly increased relative hydraulic conductivity (100 cm^3/ft.) and decreased excess pore pressure (+0.2 kg/cm^2). The only inconsistent parameter was friction ratio which continued to decrease in a linear fashion.

---

[32] At the Gilson Road site a great deal of quality control was exercised to ensure excavation to the top of bedrock. Quality control procedures included excavation until no additional material could be removed, verification as based on borings at 100 to 250 foot spacing, and re-excavation just ahead of the advancing backfill face to clean the key (Ayres, 1983). During excavation, bedrock, as identified via fracture plane and lithology, was typically removed from the bottom of the trench for visual inspection. However, it is likely that occasional small pockets of glacial till remained in the smaller bedrock convolutions after backfilling. The specification required air lifting to clean these pockets but it was not enforced due to the enhanced potential for side wall instability which air lifting would induce.

The piezocone data, in and of themselves, indicate that the cutoff wall was not constructed to the top of bedrock. As based on a penetration of this material of over 6 feet, the data further indicated that bedrock is at least[33] that distance below the wall. However, another explanation for the data exists which is more consistent with the quality control data obtained during construction as well as the piezocone data. It is hypothesized that the sounding penetrated the side of the cutoff wall and then proceeded into the native formation. It is further hypothesized that this was due to a non-vertical cutoff wall rather than an inclined sounding[34]. The following data support this conclusion:

---

[33] Point resistance did not indicate bedrock was encountered. The sounding was terminated when enough down pressure was applied to lift the drill rig off the ground. This is not reflected in the point loads because a significant portion of the penetration resistance was likely to have been mobilized at the enlarged shoulder corresponding to the drill string (see Figure 3-2).

- After excavation, the slurry trench was sounded to determine its depth and a measurement of 102 feet was obtained at station 1+00. This depth corresponded to the top of bedrock as verified via the above mentioned quality control measures. These data indicate that the trench was in fact excavated to the top of bedrock.

- Post-construction verification borings also penetrated the sidewall of the trench prior to reaching the bottom of the wall. It is highly likely that these boreholes are very nearly vertical (see Section 3.5).

- A general trend of the linearly increasing point resistance and linearly decreasing friction ratio are, as would be expected, based on exit of the side wall at a high penetration angle (slow increase in confinement with distance from the cutoff wall).

- The two high peaks in the point resistance, relative hydraulic conductivity and excess pore pressure, along with the intervening more moderate zone, are postulated to indicate that the piezocone was moving in and out of the backfill/filter cake/native formation due to irregularities in the side wall.

- Approximately 3 feet prior to encountering a point resistance indicative of a H.D.C.G. window, the relative hydraulic conductivity index increased somewhat from values associated with the intact backfill. It is postulated that this reflects the proximity or partial contact of the porous stone with the native formation.

- Most of the other containment wall soundings also encountered native formation materials prior to reaching bedrock depth.

---

[34] While special procedures were followed to obtain vertical soundings, wall verticality was only specified to within 1 percent. This could result in a translation of about 1 foot at the bottom of the wall. However, the actual verticality of the wall was governed by how level the backhoe tracks were positioned on the working surface. A 6 inch error in the level of the working mat, over the track width, would result in a translation of 2.5 feet at the bottom of the wall.

The above data does not support the conclusion that the material encountered below 88.5 feet represents unexcavated native material. However, it is recognized that another possible explanation encompasses a window composed of native material which sloughed off the side wall "in mass" and filled the bottom of the trench prior to backfill placement. However, the available data also does not support this scenario to an extent which would render it plausible.

Independent of the exact nature and cause of the "potential window"/sidewall penetration identified below 88.5 feet in sounding GZ-13, the individual piezocone indices obtained from this location exhibit a significant degree of consistency. In addition, the magnitude and direction (positive vs. negative) of the deviations from intact backfill indices agree with theoretical trends that would be expected for a noncohesive, highly permeable material. This data demonstrates the potential usefulness of the piezocone for window detection.

B. Low Density Clean Granular Window

The occurrence of this type of window is most likely to be the result of side wall material which had sloughed onto the advancing backfill face. Two such zones were initially identified in sounding GZ-13. The first zone, located from 17.5 to 18.5 feet deep in the wall exhibited the point resistance, friction ratio and excess pore pressure signature characteristics indicative of the L.D.C.G. windows encountered in the test section. However the relative hydraulic conductivity index did not support the classification of this zone as a potential window. It was therefore dropped from further consideration due to the definitiveness of this index.

The second potential L.D.C.G. window was encountered at a depth of 43 to 48 feet. In this case, the point resistance abruptly increased to about 20 kg/cm^2 and remained relatively constant at an average value of 10 to 15 kg/cm^2. Average point resistances immediately proceeding and following this zone were commensurate with intact backfill (2 kg/cm^2). The friction ratio decreased from about 3 percent to values approaching 1 percent in this zone. Excess pore pressure decreased abruptly from an average of +1.0 kg/cm^2 to an average of 0.0 kg/cm^2 and then restabilized at 1.0 kg/cm^2 upon exiting this zone. Finally, the relative hydraulic conductivity index increased to about 30 $cm^3/ft.$ in this zone. This index yielded values of 8.7 $cm^3/ft.$ and 11.5 $cm^3/ft.$ prior to and after encountering this zone, respectively.

The above data is internally consistent and follows trends exhibited in the test section. As such, it indicates the presence of a potential L.D.C.G. window. Interpretation in this instance is not complicated by other significantly plausible explanations for the data. In fact, the identification of intact backfill both above and below the potential window add credibility to this interpretation.

C. Slurry Filled Granular Window

Three zones were encountered in sounding GZ-13 which initially indicated the potential for the presence of a S.F.G. window. The first case occurred at a depth of 28.0 to 29.5 feet. This zone yielded a characteristic signature with respect to point resistance, local friction, friction ratio and excess pore pressure. However, no deviation in relative hydraulic conductivity from intact backfill values was detected. This zone was therefore omitted from further investigation.

The second zone occurred at a depth of 88.5 feet, just above the potential H.D.C.G. window previously discussed. It is likely that the response is indicative of penetration of the filter cake at the side of the cutoff wall as indicated by the very high friction ratio spike. This zone is also not considered in more detail herein, due to the complications addressed above for the H.D.C.G. window.

The third zone of interest was located at a depth of 58.5 to 59.5 feet. This zone yielded piezocone indices which were nearly identical to the signature observed during test section soundings. The local friction value increased to about 0.4 kg/cm^2. The point resistance peaked at about 15 kg/cm^2 in the middle of this zone. This yielded the double peak in friction ratio (6 to 10 percent) encountered in the test section soundings. Excess pore pressure dropped to a near zero magnitude and relative hydraulic conductivity peaked at 40 $cm^3/ft.$ These indices yielded values commensurate with the intact backfill both before and after penetration of this zone. As such, this zone has a high probability of being a slurry filled granular window.

3.5 Window Verification

The primary piezocone indices of point resistance, local friction and excess pore pressure indicated that a significant number of potential windows existed in the containment wall. However, a relative hydraulic conductivity index commensurate with intact backfill allowed most of these potential windows to be dropped from further consideration. The next step in the window verification procedure encompassed obtaining samples of the remaining potential windows for visual classification and laboratory testing. These samples were obtained using standard split spoon and Osterberg undisturbed Shelby tubes in boreholes supported with heavy weight drilling fluid[35].

A total of twelve borings were executed in the containment wall. These borings were located in areas exhibiting the highest probability of containing windows as based on the QC data obtained during construction and the post-construction piezocone soundings. The locations of these borings are presented on Figure 3-9. As shown on the figure, some locations contain multiple borings in a relatively concentrated area. This reflects two basic problems that were encountered when attempting to obtain samples from the wall. The first concerns locating the boring at the centerline of the wall. Although the as-built survey of wall location was sufficiently accurate for subsequent construction at the site, it was not accurate enough for wall sampling. Initially, ground penetrating radar (GPR) was employed to find wall centerline. The location of the top of the wall was easily identified. However, the top of the wall was often 8 to 10 feet wide for the first 5 to 10 feet of depth and then narrowed to the standard 3 to 4 foot width[36].

The GPR could not penetrate through this upper zone and therefore could not accurately identify wall centerline. Hence, the borings sometimes missed the cutoff wall and required relocation. The second difficulty concerns wall verticality. As discussed in Section 3.4.2, it is hypothesized that the cutoff wall is not completely vertical. This has no impact on the effectiveness of the wall. However, due to the verticality of the borings[37], many of them penetrated the side of the trench and proceeded into the native formation before reaching the bottom of the containment wall. This also resulted in the requirement for additional borings.

---

[35] Additional information on sampling and drilling techniques can be found in Winterkorn, 1975, Pgs. 15 and 44.

[36] During excavation, the top 5 to 10 feet of the trench was typically sloped back on a 30 to 45 degree angle from vertical. This was done to prevent surface materials at the edge of the trench from raveling in and potentially resulting in windows.

During the sampling program, a total of fifty-four split spoon samples and fifty six tube samples were obtained. The split spoon samples were used for visual classification and as an aid in determination of tube sampling intervals. The tube samples were used to provide undisturbed material for triaxial hydraulic conductivity testing as well as visual classification. Prior to sample designation for triaxial testing, the unopened tubes were radiographed to ensure that representative samples were selected[38].

3.5.1 Sample Classification

Six of the samples retrieved from potential window locations yielded material which, if continuous through the wall, would result in actual windows. These locations are:

- Boring GZU-13, cutoff wall station 1+00 at a depth of 43 to 48 feet.
- Boring TBF-1A, cutoff wall station 3+45 at a depth of 39 to 46 feet.

---

[37] The borings were advanced using drilling fluid supported, open-hole techniques. Large diameter drill rods (NX) were used and the rigs were precision leveled prior to and during the drilling. The combination of the leveling, the weight of the rods, and the soft consistency of the cutoff wall backfill foster a high degree of verticality. These techniques, as used on other projects where the boreholes were surveyed with inclinometers, typically result in a maximum error of 3 to 6 inches at the bottom of 100 foot deep bore holes.

[38] Radiography allows general classification of the soil in the Shelby tubes prior to opening them. The method is based on x-ray transmission characteristics of the tube contents. The procedure proved valuable as a tool to identify representative portions of the samples for triaxial testing, particularly with respect to intact backfill. Samples containing large pieces of gravel could thus be rejected from testing even when the gravel was not visible at the surface of the sample. Samples containing large pieces of gravel (greater than one-sixth the sample diameter) can yield artificially low values of hydraulic conductivity.

- Boring SU-7, cutoff wall Station 33+30 at depths of 30.5 to 35.5 feet, 45.5 to 48 feet, and 49.5 to 52 feet.

- Boring SU-4, cutoff wall Station 37+50 at a depth of 21 to 26 feet.

Each of the above potential windows are discussed individually in the following subsections. In addition to the above enumerated potential window samples, two cases were encountered where a sample could not be retrieved from a potential window location. These were boring GZ-13U, 58.5 feet to 59.5 feet, boring TBF-1A, 9.5 feet to 10.5 feet. As such, a final determination of material classification could not be made. In cases where the borehole encountered dense native formation materials above bedrock, the available data indicate that this was the result of trench sidewall penetration due to non-verticality of the cutoff wall rather than unexcavated material at the bottom of the wall. Samples of this material are therefore not treated further herein.

3.5.1.1 Boring GZU-13, 43 to 48 Feet

This potential window was identified at station 1+00 in sounding GZ-13. The boring, located 5 feet down-station from the sounding, encountered the potential window at a depth of 48 feet. Two samples of the potential window were obtained. The first sample was a split spoon from 48 to 50 feet and the second was a tube from 50 to 52 feet. The samples were classified as a loose, light brown, coarse to fine sand, trace fine gravel, trace silt, in accordance with the Burmister system. This material is consistent with the piezocone identification as a low density clean granular window.

After the above two samples were obtained, the boring was advanced and remained in granular deposits until refusal was encountered at a depth of 90 feet. The deeper granular deposits appear to be the result of penetration of the trench sidewall. Inasmuch as additional backfill was not encountered below the potential window, the identification of this material as a window is inconclusive. In fact, this material may also correspond to sidewall penetration rather than the zone identified in sounding GZ-13.

3.5.1.2 Boring TBF-1A, 39 to 46 Feet

This potential window was identified at station 3+45. The window consisted of two separate materials. The upper 4.5 feet contained a loose, light brown, coarse sand, trace medium sand, trace silt. The material exhibited a uniform grain size and the voids were filled with bentonite slurry. As such, this material corresponded to a slurry filled granular window which was caused by sedimentation of material suspended in the support slurry.

The lower 2.6 feet of this window contained a medium dense, gray, fine to coarse sand, trace gravel, trace silt. It appears that this material was glacial till which was located in a bedrock surface convolution and therefore could not be excavated.

Boring TBF-1, located 5 feet down-station of TFB-1A also encountered this window. In this case, the window was 6.5 feet thick. Sounding GZ-12, located an additional 5 feet down-station of TBF-1A, did not encounter the window. In the up-station direction, sounding GZ-11, located 5 feet from TBF-1A, encountered the window over a thickness of 3.5 feet. This data indicates that the window is greater than 10 feet long in the direction of the cutoff wall alignment.

3.5.1.3 Boring SU-7, 30.5 to 35.5 Feet, 45.5 to 48 Feet and 49.5 to 52.0 Feet

These three potential windows were initially identified at station 33+30 in sounding S-7. The piezocone signatures for each of the three windows were very similar. They each exhibited significant point resistance similar to a L.D.C.G. or S.F.G. window (15 to 20 kg/cm^2 as compared to 5 kg/cm^2 for the intact backfill). Local friction values were also relatively high as would be expected for a S.F.G. window (0.4 to 0.6 kg/cm^2). These indices yielded friction ratios which averaged from 3 to 5 percent with local peaks as high as 7 percent. Although the magnitude of the indices are similar to those expected for a S.F.G. window as based on the test section soundings, the friction ratio signature did not exhibit the characteristic shape. Instead of the well defined friction ratio spikes upon entering and exiting the window, friction ratio in this case yielded consistently high values. This behavior would be expected if the piezocone encountered the filter cake at trench sidewall irregularities. This hypothesis can not be further supported by the piezocone data in that excess pore pressure and relative hydraulic conductivity data was not available.

Boring SU-7 was located 5 feet down-station of the sounding. Sampling confirmed two zones corresponding to the above potential windows. The depths of these zones were 29.0 to 35 feet and 48 to 50 feet. As such, they are somewhat displaced from that indicated by the sounding. The samples were classified as loose, brown, coarse to fine sand, little fine gravel, trace silt. Bentonite slurry was not noted in the soil pore space. These samples were obtained via split spoon. Tube samples were attempted but they yielded no recovery.

The samples support classification as a L.D.C.G. window. However, this window type is not corroborated by the piezocone data. As with the piezocone sounding, it is probable that the boring penetrated into native formation material at locations of irregularities in the trench sidewall. This hypothesis is supported by:

- A material type which corresponds to native formation deposits.
- Penetration of the sidewall below a depth of 52 feet. The boring thereafter remained in native formation material until refusal at 60 feet.
- Piezocone response uncharacteristic of window types anticipated in the cutoff wall but consistent with partial sidewall penetration.
- An identical piezocone response in sounding S-9 from 23 to 28 feet. Boring SU-9 encountered the same zone at 22 feet but remained in the native formation until refusal at 30 feet.

The data is inconclusive as to the cause of the non-backfill material encountered. While the existence of some type of window not modeled in the test section is certainly possible, it is more likely that the sounding and boring both penetrated irregularities in the sidewalls of the trench. Such irregularities are often observed in concrete backfilled slurry trenches when the soil is removed from the walls. Irregularity magnitudes of 1 to 3 feet are not uncommon, particularly in granular formations.

3.5.1.4 Boring SU-4, 21 to 26 Feet

This potential window was identified at Station 37+50 in Sounding S-4. The piezocone signature in this case was identical to the three zones described in Section 3.5.1.3 (Sounding S-7).

As such, it is hypothesized that Sounding S-4 also penetrated an irregularity in the trench sidewall or initially penetrated the sidewall and was deflected back into the soft backfill by the denser native deposits. Boring SU-4, located 5 feet down-station, did not encounter these potential window materials. Continuous samples were taken starting 3 feet above the top of the potential window and proceeded 4 feet below it's bottom. Only intact backfill was recovered therefore indicating that the boring did not encounter the trench sidewall.

3.5.2 Sample Hydraulic Conductivity Testing

In only two cases could undisturbed samples of potential windows be obtained. These were from boring GZU-13 and TBF-1A. While triaxial hydraulic conductivity tests could have been performed on those samples, the effort was judged unnecessary. The grain size distribution of the material in the samples indicated a probable hydraulic conductivity of 10^{-2} to 10^{-3} cm/sec. Further refinement of this estimate provides no significant information in that this value is high as compared to the design specification for the cutoff wall of 1×10^{-7} cm/sec. Therefore, if these materials exist in a continuous state transecting the cutoff wall, then a window would certainly exist.

Investigation with respect to the hydraulic conductivity of undisturbed intact backfill samples was considered a more valuable endeavor. As such, representative tube samples of intact backfill were subjected to triaxial testing. The results of this work and the conclusions drawn from the data are presented in Section 2.

3.5.3 Containment Wall Homogeneity

One of the objectives of the containment wall soundings was to provide an estimate of the probable frequency of window occurrence in the cutoff wall. Nearly 1,500 feet of sounding and borehole installation yielded only two cases where the probability of the existence of a window was high. The first case was boring GZU-13 as described in Section 3.5.1.1. In this case, window identification although probable, was inconclusive due to the lack of an intact backfill sample below the potential window. However, it is believed that the second case (boring TBF-1A, section 3.5.1.2) was conclusively identified as a non-backfill deposit located between the bottom of the cutoff wall and the bedrock. In this case, the same undisturbed tube sample contained intact backfill directly above S.F.G. window material. It is unlikely that this data fits a plausible non-window explanation.

The above data indicated that few windows are likely to exist in the Gilson Road containment wall. However, it is recognized that much of the sounding data was inconclusive due to complications arising from trench side wall penetration. While this hypothesis is probable, it can not be proven with the available data. The inability to adequately sound and sample the containment wall was a direct result of the unknown location of exact wall center line and the probable non-verticality of the wall. While these issues do not result in deficiencies with respect to containment performance, they do render post-construction verification a very difficult, expensive and inclusive undertaking. As such, greater quality control should be exercised in these areas.

Finally, it is emphasized that even in the case where the existence of a window was highly probable, its impact on the efficiency of the cutoff wall can not be determined via sample retrieval. Sampling provides no indication of the degree to which the window material is continuous through the wall. In fact, it is probable that under the worst case conditions, a filter cake exists between the window material and the native formation. A filter cake, such as would be anticipated, would render the hydraulic conductivity of the overall wall cross-section orders of magnitude below that of the native formation. The effectiveness of the overall cutoff wall can only be evaluated by hydraulically stressing the wall and observing the piezometric response of the system. This task is treated in Section 4.

3.6 Conclusions

The following conclusions can be drawn from the data presented herein:

- The test section did not provide the known condition expected for piezocone performance evaluation. The lack of support slurry filled window long-term stability provided valuable insight into the improbability of the existence of such a window in the containment wall. However, the inability to construct other window types to a known geometery impaired verification of piezocone suitability for window detection in soil/bentonite backfilled cutoff walls.

- The response of the piezocone when encountering windows in the test section was in general accordance with that anticipated based on prior empirical data and theoretical considerations. As such, window locations could be identified in the test section. The windows produced repeatable and identifiable signatures in the piezocone indices. Based on the test section soundings, it appears that the piezocone is capable of window detection in soil/bentonite backfilled cutoff walls.

- As an adjunct to the standard piezocone indices, the relative hydraulic conductivity index provided valuable additional insight with respect to potential window identification. To enhance the utility of this system, redesign should be undertaken to allow high flow volumes at low driving pressures. This would better allow evaluation of the hydraulic significance of a potential window while minimizing preferential boundary flow and hydraulic fracturing.

- Potential window identification must utilize a multi-index analysis. Many cases were found in which one or two indices indicate window existence but the other indices did not. In particular, the relative hydraulic conductivity index often demonstrated that a high hydraulic conductivity zone was not present in locations initially identified as potential windows based on the other indices.

- While anomalies in the cutoff wall backfill were relatively easy to identify as based on piezocone response, significant judgment was required to identify characteristic responses which were likely to be associated with potential windows. This judgment must be based on a thorough understanding of piezocone operational principles as well as the stress, strain, hydraulic conductivity characteristics of soil deposits.

- Based on the data acquired during this portion of the project, it appears that the piezocone could provide a useful, routine post-construction verification capability. However, additional work must be conducted to build up a data base upon which empirical correlations can be constructed. This magnitude of data generation would likely require use of the instrument on a number of different cutoff wall projects following additional controlled test section sounding.

- For piezocone sounding to be viable as a post-construction verification tool, construction quality control of the cutoff wall must include wall alignment and verticality provisions. The accuracy of as-built wall alignment documentation and actual wall verticality must be increased significantly above that required for hydraulic performance only.

- The piezocone soundings and undisturbed sampling of the Gilson Road cutoff wall encountered windows in only a few instances. These investigations were concentrated in areas with the highest probability of window occurrence as based on construction quality control data. This data therefore reinforces the conclusion that the cutoff wall behaves essentially as an intact barrier. However, it is recognized that the data is somewhat inconclusive due to complicating factors associated with the trench side wall penetration issue. Sidewall penetration was likely due to the probable non-verticality of the cutoff wall.

SECTION 4

EVALUATION OF CONTAINMENT LEAKAGE

Passive physical containment (in-situ impervious barriers) forms an integral part of the overall containment strategy adopted for the remediation of the Gilson Road site. The three major components of the passive containment system are:

1. The synthetic surface cap which acts as a barrier to the vertical infiltration of precipitation and thus forms the top of the containment vessel,

2. The soil/bentonite cutoff wall which impedes the horizontal flow of groundwater into and out of the site and acts as the sides of the containment, and

3. The natural glacial till/bedrock aquiclude which forms the bottom of the containment.

Imperfections in any one of these three components may result in leakage and thus off-site migration of contaminants. From the standpoint of environmental damage and human health risk, if there is a breach in the containment, it may not be particularly important which element is leaking. However, from the standpoint of evaluating the inherent viability of a specific containment technology or determining an appropriate hydrodynamic isolation system (active physical barrier) to augment the passive containment, as was the case for the Gilson Road site, it is critical to assess the integrity of each component individually.

The objective of this, the third phase of the project, was to evaluate the hydraulic effectiveness of the soil/bentonite cutoff wall. The construction quality control testing and the post-construction wall sampling/testing (Phase One, as reported in Section 2.0) demonstrated that the intact hydraulic conductivity of the cutoff wall generally met the contract specification of 1×10^{-7} cm/sec (3×10^{-4} ft/day). However, the effectiveness of the barrier is proportional to the cutoff wall bulk hydraulic conductivity, of which intact hydraulic conductivity is only a part. Ideally, the evaluation of bulk cutoff wall hydraulic conductivity would be independent of the influence of the other barriers. Inasmuch as the cap is located in the vadose zone, it only indirectly influences groundwater flow. Its effect on the analysis of bulk cutoff wall conductivity can therefore be easily addressed. Unfortunately, both the cutoff wall and aquiclude directly impact groundwater movement, and component separation in this case is more difficult.

Two approaches were used to separate the cutoff wall and aquiclude, such that the effectiveness of the cutoff wall alone could be evaluated. The first approach was based on the use of a 3-dimensional numerical model of the site which was previously used to design the hydrodynamic isolation/recirculation system for the project. The first step in this approach was to better evaluate the aquiclude hydraulic conductivity via a bedrock pumping test (Section 4.1). The second step was to employ a numerical sensitivity analysis based on vertical piezometric head distributions to evaluate bulk cutoff wall hydraulic conductivity (Section 4.2) The second approach consisted of evaluation of contaminant concentration versus time and depth (Section 4.3).

4.1 Bedrock Pumping Test

The primary objective of the bedrock pumping test was to evaluate the hydraulic conductivity of the fractured rock which forms the bottom of the containment below the cutoff wall. Direct determination of bedrock hydraulic conductivity was an essential step in the evaluation of the bulk hydraulic conductivity of the cutoff wall. Particular emphasis was placed on determining the bedrock hydraulic conductivity in the direction perpendicular to the cutoff wall. This value would control the leakage under the wall should the bedrock fracture pattern exhibit anisotropic characteristics.

Documentation of the hydraulic conductivity of the rock has also increased confidence in the calibration of the numerical model which was used to design the hydrodynamic isolation system. During model calibration, it was necessary to use conductivities for the bedrock which were significantly greater than those measured during packer testing. The "unjustifiably" high values were required to match vertical heads simulated by the model to the field observations. The pumping test has demonstrated that these higher conductivities were essentially correct.

The secondary objective of the pumping test was to assess the degree of hydraulic interconnection between the fractured rock and the high hydraulic conductivity sand and gravel deposits of the upper aquifer. This was achieved via evaluation of an effective homogeneous leakance[1] for the glacial till aquitard.

4.1.1 Pumping Well Location and Installation

The pumping well, PT-1, was located at the western end of the site as shown in Figure 4-1. There were several considerations involved in this selection. First, this portion of the site experiences the greatest head loss across the cutoff wall. Therefore, the bedrock hydraulic conductivity at this location more strongly affects leakage out of the containment, particularly because this is the downgradient end. Second, the glacial till thickness increases in this area, providing greater isolation between the bedrock and overburden aquifers. Therefore, evaluation of bedrock hydraulic conductivity is simplified and confidence in the evaluation is increased. Third, the downgradient portion of the cutoff wall would be the most highly stressed during the subsequent overburden test, thereby allowing better comparison between the two tests. Finally, a greater abundance of monitoring instrumentation was already in place in the vicinity of the downgradient portion of the wall.

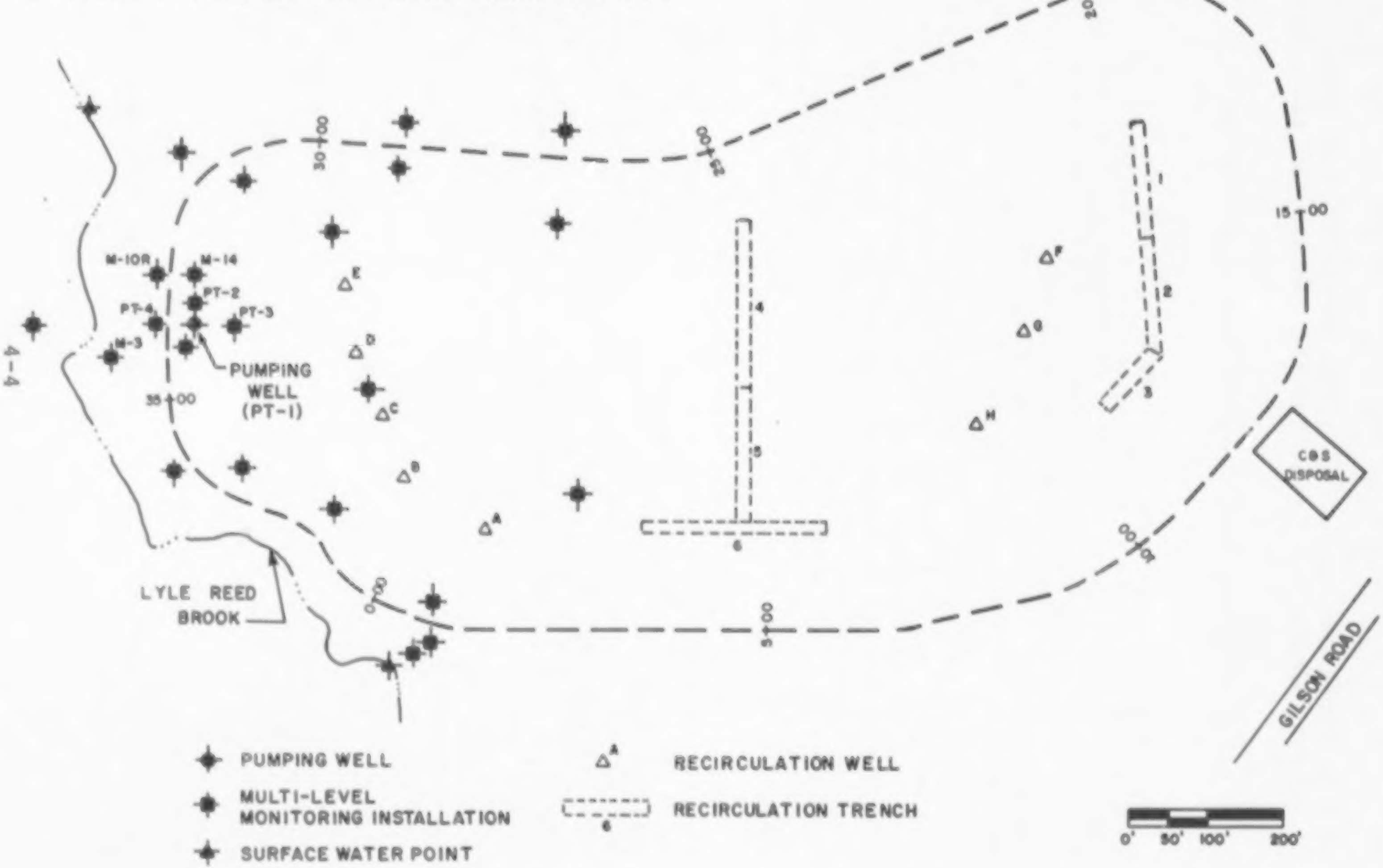
The pumping well was placed inside the cutoff wall so as not to draw contaminants outside of the containment during execution of the test. At the downgradient portion of the site, the bedrock dips sharply in the direction perpendicular to the wall. Therefore the well was placed close to the cutoff wall so as to stress those materials which most directly affect leakage rates, i.e., those immediately below the bottom of the wall. In addition, the horizontal component of flow induced in the overburden would be roughly radial, and therefore parallel to the low hydraulic conductivity barrier. This minimized the inaccuracies associated with approximating the overburden aquifer as homogeneous in the mathematical analysis.

The objective of the well installation was to allow pumping from the bedrock only, and therefore the overburden and glacial till were sealed off. The entire column of rock was to be stressed, down to at least 50 feet below the glacial till, in order to include the major fracture zones. Earlier drilling at well R-1, including continuous coring and classification by RQD

---

[1] Hydraulic resistance to vertical flow between aquifers separated by an aquitard.

FIG. 4-1 PUMPING TEST MONITORING NETWORK


30+00
25+00
20+00
15+00
10+00
5+00
0+00
35+00
M-10R
M-14
PT-2
PT-4
PT-3
M-3
PUMPING WELL (PT-1)
E
D
C
B
A
F
G
H
1
2
3
4
5
6
LYLE REED BROOK
CBS DISPOSAL
GILSON ROAD
PUMPING WELL
MULTI-LEVEL MONITORING INSTALLATION
SURFACE WATER POINT
RECIRCULATION WELL
RECIRCULATION TRENCH
0'
50'
100'
200'

(rock quality designation[2]), had shown that, in addition to a highly fractured upper zone, a 5 foot thick highly fractured zone (RQD = 20%) existed at a depth of 50 feet. Therefore the possibility existed that leakage was occurring at significant depths, and not merely in the highly fractured upper portion of the bedrock. The depth of the borehole was therefore established to penetrate 100 feet into the bedrock.

A 6-inch casing was installed through the overburden, including the glacial till, and drilling of the bedrock began using the air-rotary method. However, the top 10 feet of the highly fractured rock was unstable and would not remain open. Therefore the casing was driven through this layer as well. The hole was then advanced uncased through 89 feet of mostly competent rock. Finally, the lower 10 feet of casing was perforated to reestablish hydraulic connection with the upper portion of the bedrock.

The hole was tested several times during the drilling process for its water yield. The water yield, or water make, was estimated by forcing air into the bottom of the borehole, and measuring the flow rate of water which was forced out of the top of the casing at steady state. Before the casing was perforated, the yield was approximately 10 gpm (gallons per minute), but after perforation the yield increased to about 150 gpm. This demonstrated the dominance of the upper zone regarding the transmissivity of the formation. In addition, the yield of 150 gpm indicated that the upper fractured zone was far more permeable than had been inferred from the previous packer test data.

After drilling and development operations were completed, the pump was placed in the casing at a depth of 75 feet. A piezometer was also included to provide water level measurements in the well. Details of the installation are shown in Figure 4-2.

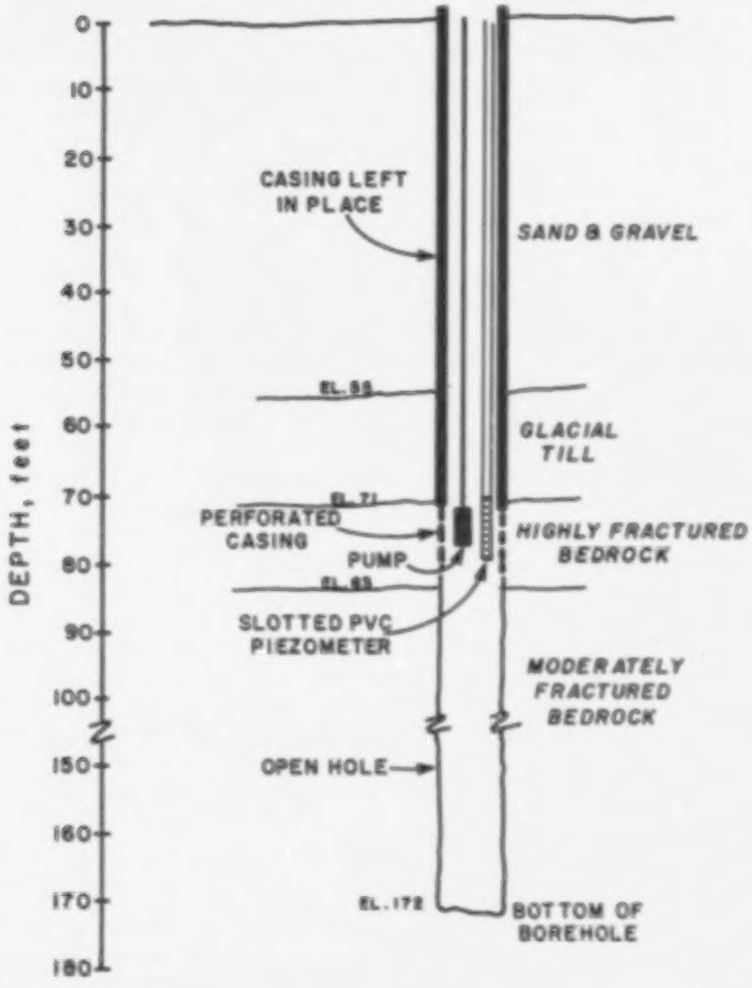
4.1.2 Recharge Trenches

Water pumped during the test had to remain on the site due to its contamination. Therefore, the water was recharged to the overburden via three recharge trenches already in place. These trenches, numbers 4, 5, and 6 as shown in Figure 4-1, were

---

[2] An index used to quantify rock mass integrity with respect to degree of fracturing.

FIG. 4-2 PUMPING WELL INSTALLATION


DEPTH, feet
0
10
20
30
40
50
60
70
80
90
100
150
160
170
180
CASING LEFT IN PLACE
SAND & GRAVEL
EL. 55
GLACIAL TILL
EL. 71
PERFORATED CASING
HIGHLY FRACTURED BEDROCK
PUMP
EL. 83
SLOTTED PVC PIEZOMETER
MODERATELY FRACTURED BEDROCK
OPEN HOLE
EL. 172
BOTTOM OF BOREHOLE

already operating as part of the hydrodynamic isolation system. The pumpage from the bedrock pumping test was added to the existing flow.

4.1.3 Monitoring Points

Drawdown resulting from pumping the bedrock well was measured in the pumping well and 72 pre-existing and recently installed piezometers and observation wells. Observation points were divided among 24 borings, most of them multi-level installations (see Figure 4-1). Specific emphasis was placed on measurements in the upper rock layer and in the overburden just above the glacial till. The upper, highly fractured portion of the bedrock was of primary interest in that it controlled leakage under the cutoff wall. Measurements just above the glacial till were also emphasized inasmuch as the difference in drawdowns across the glacial till controls the amount of induced leakage through the aquitard and thus recharge to the bedrock.

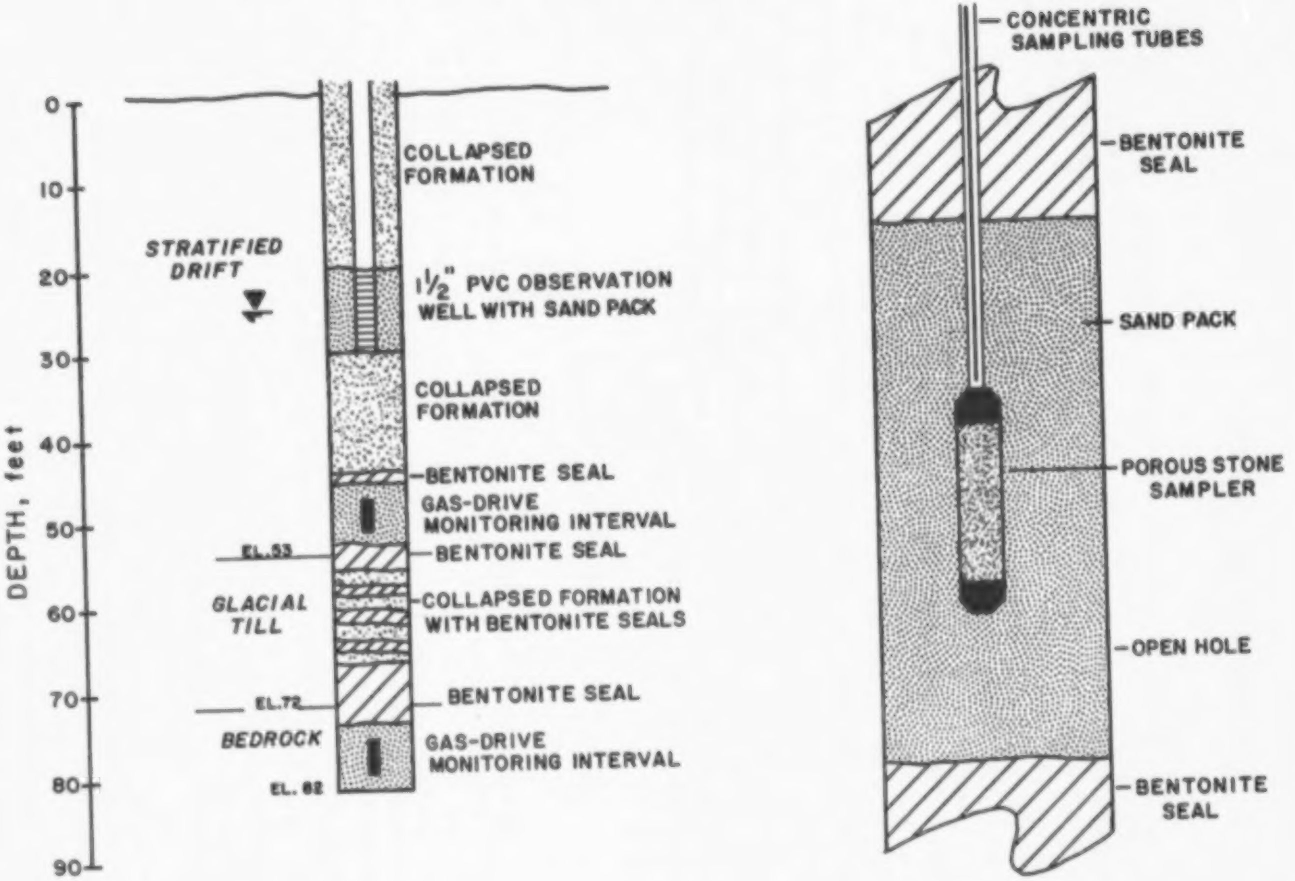
The locations of monitoring points ranged horizontally from as close as 10 feet to as far as 500 feet away from the pumping well, and vertically from the water table to 100 feet below the top of bedrock. In addition, water levels were monitored at five points in nearby surface waters.

4.1.3.1 Existing Points

Prior to the pumping test, numerous observation wells and gas-drive sampler/piezometers (Barvenik, 1983) existed at the site as the result of previous engineering studies (e.g., design and operation of hydrodynamic isolation systems for emergency response and final remediation). In particular, there were numerous multi-level installations both inside and outside the cutoff wall for the purpose of estimating leakage through the site. These monitoring installations typically included a 1-1/2 inch PVC observation well at the water table, one or two sampler/piezometers in the stratified drift, and one in the top 10 to 15 feet of the bedrock. A typical installation is presented as Figure 4-3.

The method used to construct these installations was typically as follows. The casing was driven and washed out with a roller bit down to the top of the bedrock. The hole was then advanced by coring 10 to 15 feet into the bedrock. A sampler/piezometer was placed in the bedrock inside a sand filter. The casing was then extracted incrementally as a compacted

FIG. 4-3 TYPICAL EXISTING MULTILEVEL INSTALLATION


DEPTH, feet
0
10
20
30
40
50
60
70
80
90
STRATIFIED DRIFT
GLACIAL TILL
BEDROCK
EL.53
EL.72
EL.82
COLLAPSED FORMATION
1½" PVC OBSERVATION WELL WITH SAND PACK
COLLAPSED FORMATION
BENTONITE SEAL
GAS-DRIVE MONITORING INTERVAL
BENTONITE SEAL
COLLAPSED FORMATION WITH BENTONITE SEALS
BENTONITE SEAL
GAS-DRIVE MONITORING INTERVAL
CONCENTRIC SAMPLING TUBES
BENTONITE SEAL
SAND PACK
POROUS STONE SAMPLER
OPEN HOLE
BENTONITE SEAL

bentonite seal was placed to seal the borehole at the level of the glacial till aquitard. This process was repeated for each sampler/piezometer with intermediate seals placed such that the vertical hydraulic conductivity of the installation was less than that of the parent formation. An observation well was then installed with the screen positioned so as to straddle existing and projected water table elevations.

To allow groundwater samples to be extracted from the formation, the gas-drive sampler/piezometers include one-way check-valves, which allow unrestricted flow from the formation into the access tube but which restrict, without completely preventing, flow in the opposite direction during sampling. They are therefore not capable of responding adequately to rapid drawdowns associated with pumping tests in confined aquifers. If, however, drawdowns are occurring more slowly, the instrument response to decreasing piezometric head is consistent with that of the aquifer. In particular, the check-valve does not interfere at all with steady state water level measurements in that transient effects are, by definition, absent. This will be documented in a subsequent section. In addition, the sampler/piezometers are considered satisfactory for recovery responses, where heads are rising and the check-valve is open and thus unrestrictive. In fact, response time for the sampler/piezometers is better than for standard observation wells when heads are increasing, due to the lower storage volume associated with the small diameter riser tubes (Barvenik 1984).

4.1.3.2 New Installations

Three new monitoring installations (PT-2, PT-3, and PT-4) were constructed specifically for this study. The primary reason for the new installations was to provide rapid response drawdown data at small distances from the pumping well. In addition, they were drilled to about the same depths as PT-1 in order to obtain an understanding of the importance of the deeper zones of flow in the bedrock. Two of the wells, PT-3 and PT-4, were lined up with PT-1 perpendicular to the cutoff wall, because that directional hydraulic conductivity controls the specific discharge from the containment. PT-4 was located just outside of the cutoff wall in order to better isolate the section of bedrock directly underneath the wall. PT-2 was included to estimate the degree of horizontal anisotropy.

The instrumentation differed from that in the pre-existing installations in that each sampler/piezometer was supplemented with a separate small-diameter PVC piezometer. The access tubes for those piezometers were designed specifically to accommodate pressure transducers to measure the small-time transient response of the aquifer. Transducers were required because, due to the semi-confined nature of the bedrock aquifer, drawdowns were expected to occur too quickly for manual water level measurement techniques. In addition, problems associated with the time lags due to the check-valves in the sampler/piezometers were thus bypassed.

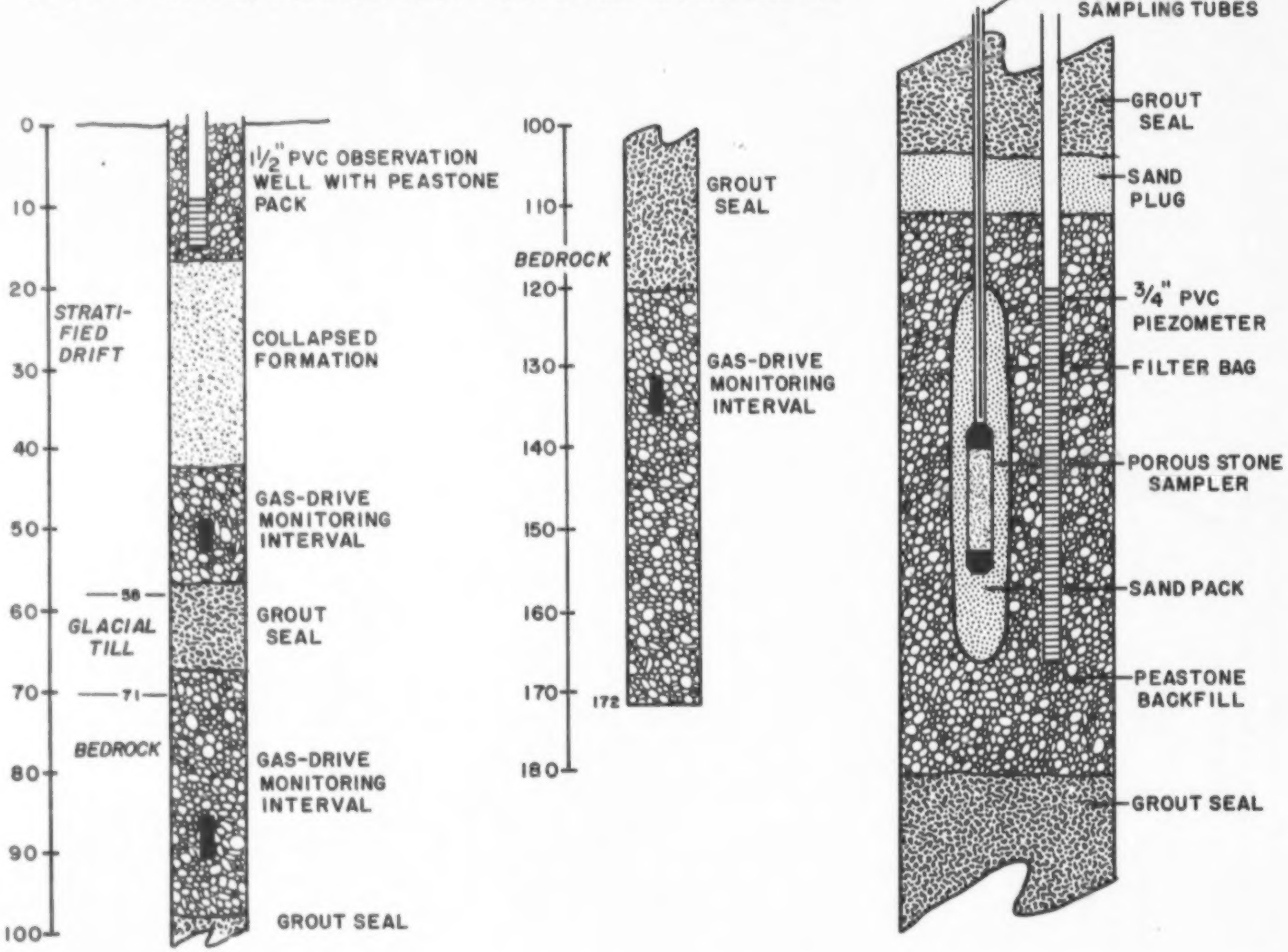
There were several objectives to be met in locating the piezometers within the new installations. The piezometers in the bedrock were to be located in the most densely fractured zones. They also required a minimum vertical separation in order to avoid redundancy and allow adequate vertical distance to establish an effective seal. Finally, measurement points were desired directly above and below the glacial till in order to assess the amount of induced leakage through the aquitard. In order to meet these objectives, one point was placed as deep as possible into the bedrock such that it coincided with a zone exhibiting a high fracture density. The second monitoring point was located in the highly fractured upper 10 feet of the bedrock. A bentonite/cement grout seal at least 20 feet long was situated in the more competent rock between these two locations. A third monitoring location was placed in the sand/gravel aquifer just above the glacial till, with a seal placed over the entire glacial till thickness. Finally a 1-1/2-inch PVC observation well was screened in the upper portions of the stratified drift phreatic aquifer. A typical installation is presented in Figure 4-4. Additional details regarding bedrock monitoring installations of this type can be found in Barvenik (1983).

4.1.4 Execution of Pump Test

4.1.4.1 General Conditions

Due to the high water yield observed in the pumping well during drilling, it was anticipated that the transmissivity of the bedrock might be higher than had been estimated from previous packer test results. Therefore, prior to execution of the full-scale test, a preliminary test was executed to determine the response time and degree of drawdown which might be expected. The response time was quite fast, indicating that transducers should be used. However, the amount of drawdown observed (less than 3 feet) was much lower than expected. The accuracy of the transducers was determined to be insufficient for monitoring the full scale test. Therefore, water level readings had to be taken manually. Electric water level readers, typically dedicated to

FIG. 4-4 TYPICAL PUMPING TEST MULTILEVEL INSTALLATION


0
10
20
30
40
50
60
70
80
90
100
STRATI-FIED DRIFT
58
GLACIAL TILL
71
BEDROCK
1½" PVC OBSERVATION WELL WITH PEASTONE PACK
COLLAPSED FORMATION
GAS-DRIVE MONITORING INTERVAL
GROUT SEAL
GAS-DRIVE MONITORING INTERVAL
GROUT SEAL
100
110
BEDROCK
120
130
140
150
160
170
180
172
GROUT SEAL
GAS-DRIVE MONITORING INTERVAL
CONCENTRIC SAMPLING TUBES
GROUT SEAL
SAND PLUG
¾" PVC PIEZOMETER
FILTER BAG
POROUS STONE SAMPLER
SAND PACK
PEASTONE BACKFILL
GROUT SEAL

each monitoring point, allowed readings as often as every five seconds. Readings were taken to within ± 0.01 feet, but the final accuracy was probably ± 0.05 feet when all potential reading errors are accounted for.

The pumping test in the bedrock was run from September 6 through September 16, 1985, for a total of ten days. Water was extracted from the pumping well in the bedrock and returned to the overburden via recharge trenches 4, 5, and 6 located approximately 700 feet upgradient. These trenches were already operating as part of the recirculation system, and the pumpage was added to the existing flow. Please note, however, that the system had been operating for several months prior to the test, and had achieved approximate steady state. Therefore, changes in piezometric head were attributed to the increment in recharge.

The pumping rate was constant throughout the test at 88 gpm. This rate was measured by adding the flows through two parallel flow meters, and was verified by Doppler ultrasonic measurements of fluid velocity in the pipe between the meters and the trenches. The pump was run on a direct three-phase 220-volt line, thereby eliminating the attendant problems of using a portable generator.

At the beginning of the test, emphasis was placed on reading water levels at those points closest to the pumping well. For these points, readings were initially taken about every ten seconds. As will be discussed subsequently, even this rapid reading schedule proved to be too slow to capture the full transient drawdown curve due to the rapid response of the bedrock aquifer. This shortcoming in the data was primarily limited to the points in the highly fractured upper zone of the bedrock immediately adjacent to the pumping well. As time progressed and the monitoring wells closest to the pumping well approached steady state, emphasis shifted to the points further away, both horizontally and vertically. Some of the furthest points did not show significant drawdowns for more than one hour after pumping began. Thus, numerous readings at the early times were not justified for these locations.

The occurrence of heavy rains during execution of the pumping test affected the drawdowns measured in the monitoring wells. Table 4-1 shows the daily total precipitation at a nearby NOAA weather data station (Pennichuck Water Works). During the test, fluctuations of up to 0.4 feet in the water level of Lyle Reed Brook rendered transient response data for the system unsuitable for analysis, as will be discussed in the following sections.

TABLE 4-1

DAILY PRECIPITATION DURING PUMPING TEST (9/85)

Date	Precipitation (in.)
9/06/85	1.30
9/07/85	1.06
9/08/85	0.00
9/09/85	trace
9/10/85	0.45
9/11/85	0.01
9/12/85	0.00
9/13/185	0.00
9/14/85	0.00
9/15/85	0.00
9/16/85	0.00

4.1.4.2 Data Discussion

Once pumping began, steady-state conditions in the rock layers were achieved in about two days. In the overburden, achievement of steady-state conditions took longer, typically on the order of seven days. At some points however, the drawdown was still increasing slowly after ten days. Figures 4-5 and 4-6 show the time-drawdown history at two multi-level installations. The first, PT-2, is representative of the majority of the data. The second, M-14, demonstrates anomalous behavior due to a previous error in location of the seal between monitoring intervals.

At PT-2, drawdowns were higher in the bedrock than in the stratified drift. The highest drawdown occurred in the upper bedrock (PT-2-3). At this point, the majority of the drawdown took place before measurements could be made (less than 5 seconds), indicating a low storage coefficient. The deep rock piezometer (PT-2-4) ultimately developed a drawdown near that of PT-2-3. However, the response time was two to three orders of magnitude larger. This indicates that the connection between points deep in the bedrock is quite tortuous, with a low hydraulic conductivity.

FIG. 4-5 PT-2 TIME-DRAWDOWN CURVE

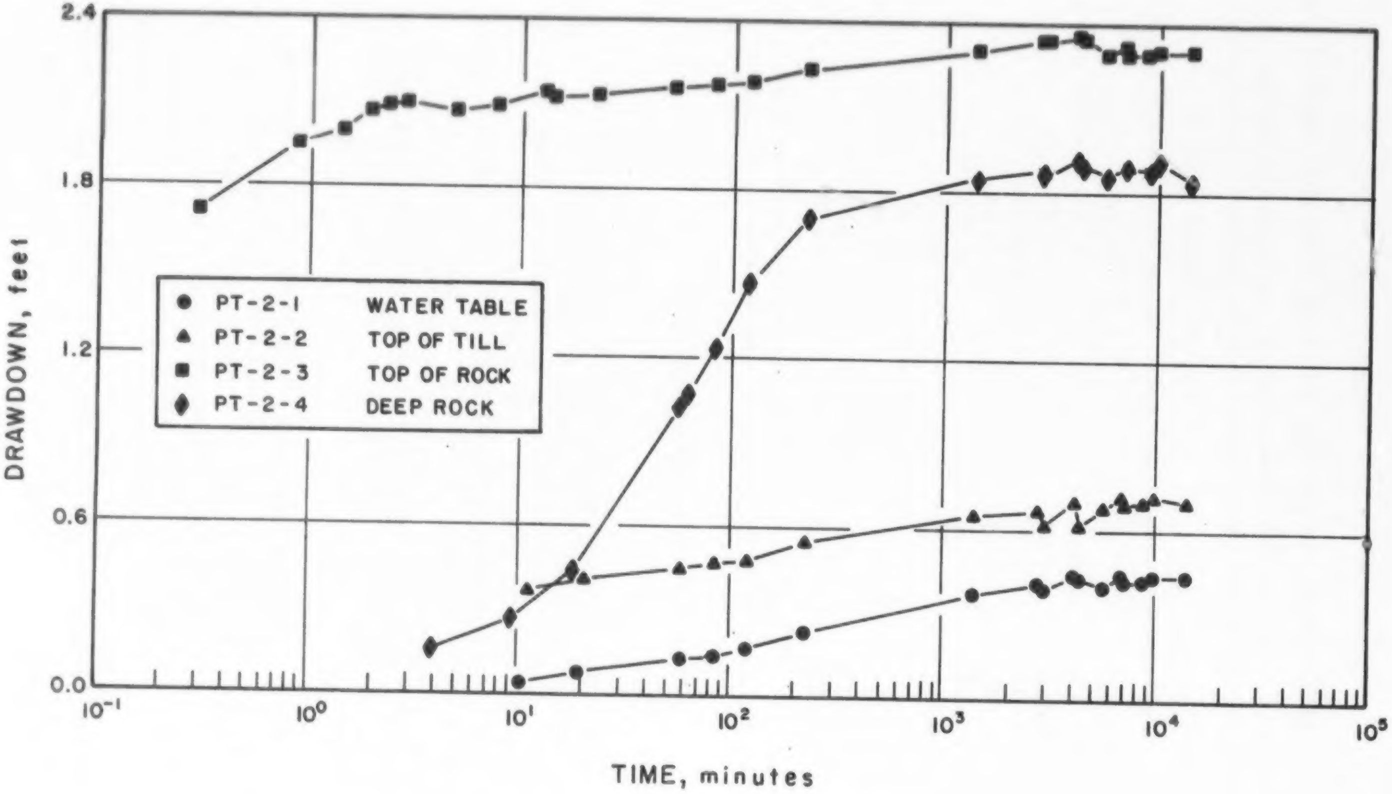
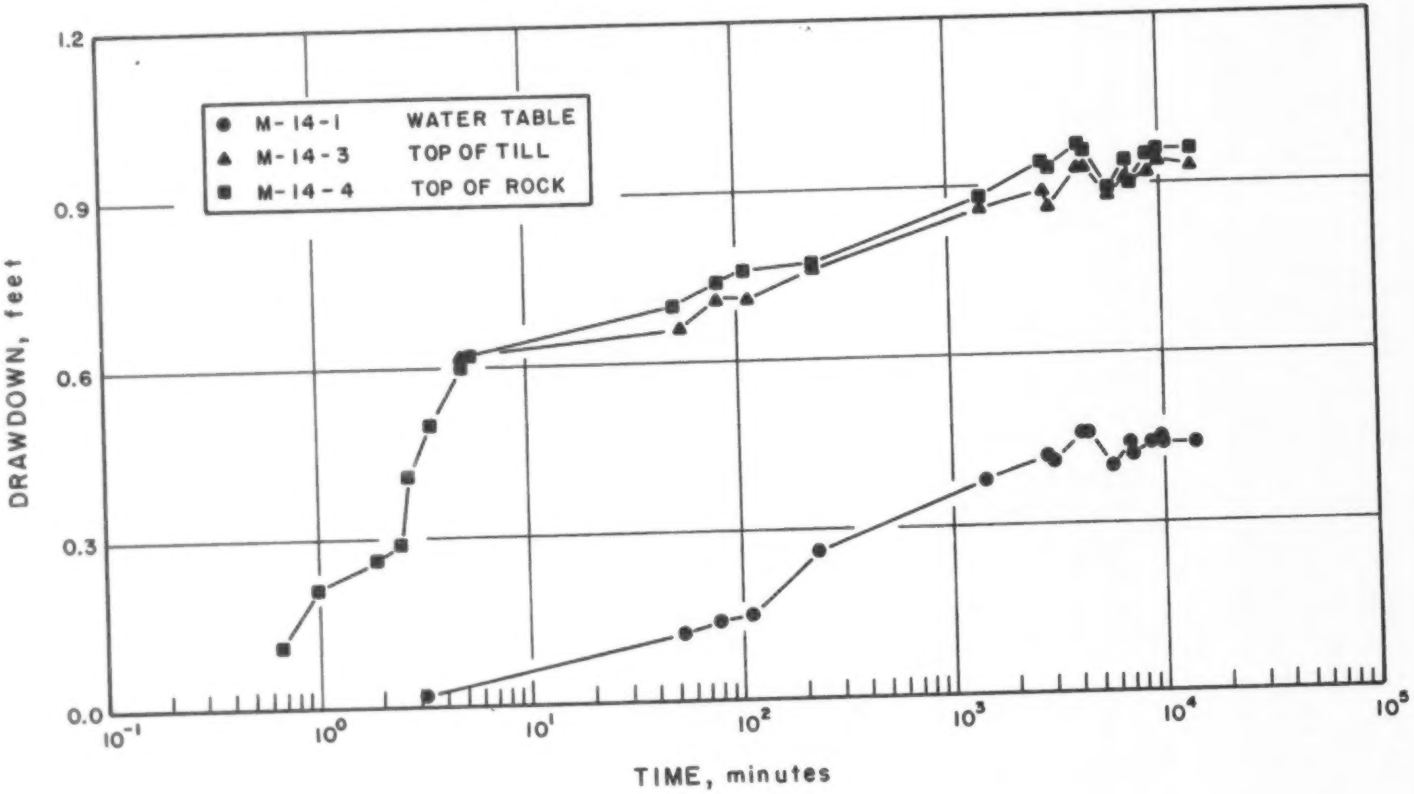


FIG. 4-6 M-14 TIME-DRAWDOWN CURVE



At PT-2-2 (above the glacial till), the drawdown was generally 2 feet less than that at PT-2-3. The small-time response for this point was not observed. The water table (PT-2-1) was the slowest to respond, since its storage coefficient is equal to the specific yield of the sands and gravels, and thus much larger than that of the lower points. The water table also experienced the least drawdown of any point in PT-2.

The induced components of flow in the aquifer near PT-2 can be deduced from the differences in drawdowns between various measurement points. The induced flow occurs from regions of low drawdown to those of high drawdown. For example, in the vicinity of PT-2, flow is occurring upwards in the bedrock and downwards in the overburden. Note that the difference in final drawdown between PT-2-1 and PT-2-2 (both in the stratified drift) is 0.3 feet, while the difference between PT-2-2 and PT-2-3 (spanning the glacial till) is 1.6 feet. This illustrates the fact that the till is much less permeable than the sands above it.

The drawdown history for M-14 illustrates the importance of seal location and integrity. M-14-4 was placed in the fractured rock zone, while M-14-3 was placed in the sand and gravel aquifer just above the glacial till. However, both points show nearly identical drawdowns. This is because the lower seal separating the two monitoring points was placed in the top of the bedrock, as is standard practice, and not in the less permeable glacial till. Another seal was placed below M-14-3 in the top of the glacial till. However, a significant section of the installation through the glacial till was not sealed. Figure 4-6 therefore illustrates the consequence of breaching the aquitard and thus establishing a connection between the two points. This small artificial leak through the aquitard probably has little effect on the overall response of the two aquifer system. However, its local effect in the vicinity of the monitoring point is significant enough to render this data misleading. This isolated case is highlighted because future sections will address the vertical head profiles at this installation.

Figure 4-7 shows the time-drawdown plot for the two monitoring points M-3-4 and M-3-5. The former is a sampler/piezometer and the latter is a piezometer only. Both were installed in the same borehole at the same depth. Note that due to its check valve, the sampler/piezometer responds more slowly at early times, but that the two points respond identically (within reading accuracy) at large times. This plot illustrates the accuracy of the gas-drive sampler/piezometers in providing steady-state hydraulic information.

FIG. 4-7 M-3-4/M-3-5 TIME-DRAWDOWN CURVE

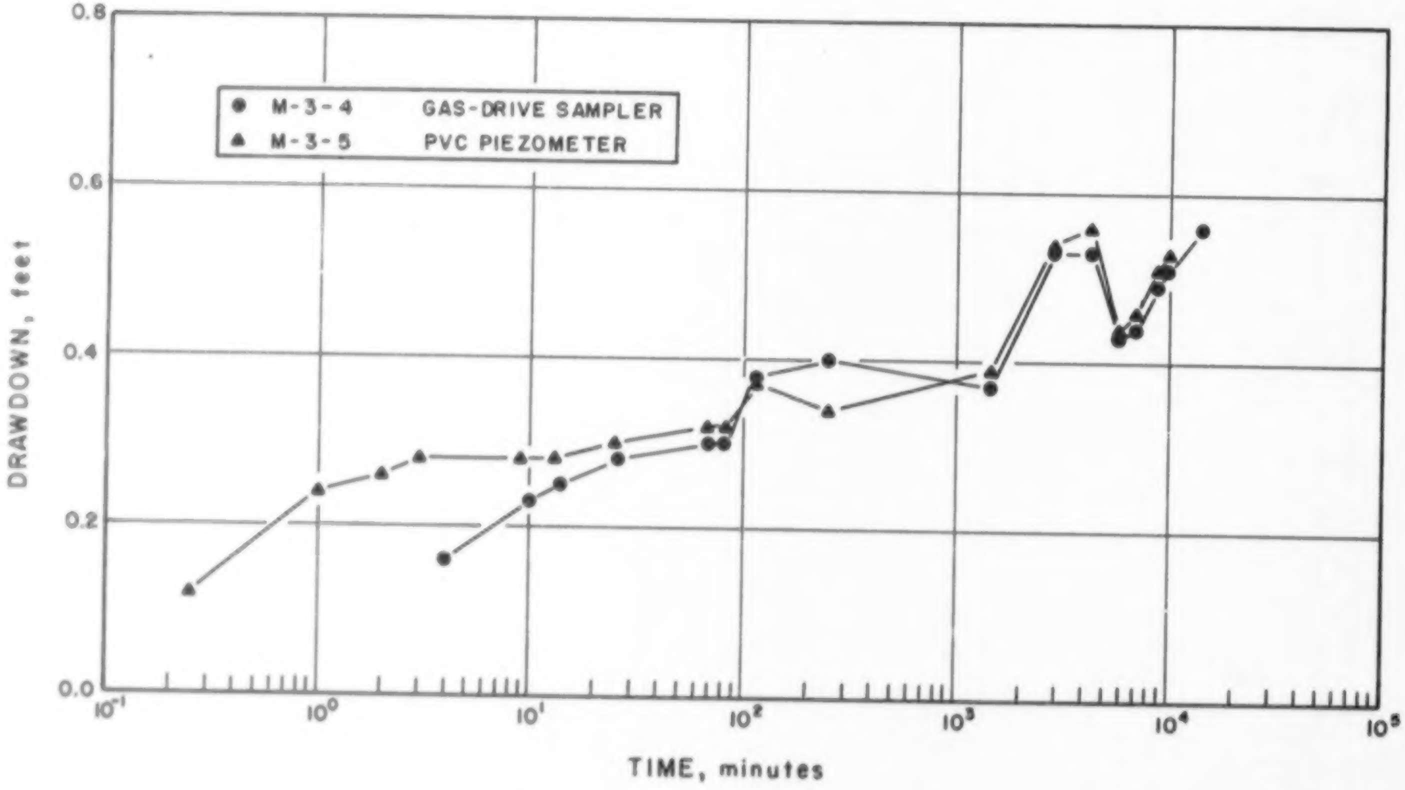


Figure 4-8 shows horizontal contours of the measured steady-state drawdowns in the fractured bedrock layer. Three features of this plot are significant. First, note that the maximum drawdown, occurring at the pumping well (PT-1), is only 2.98 feet. This is unexpectedly low, indicating a very high hydraulic conductivity. Second, the contours are nearly circular, indicating that the hydraulic conductivity is nearly isotropic in the horizontal plane. Both features strongly support the representation of this uppermost fractured bedrock zone as an equivalent porous medium (Brown and Gelhar, 1986). Third the existence of the cutoff wall appears not to have affected drawdowns in the bedrock.

Steady-state drawdowns in the overburden were vertically averaged and contoured in Figure 4-9. Drawdowns were less than 1 foot and near the recharge trenches were negative, due to the incremental mounding. The effect of the cutoff wall is seen in the 0.4 foot contour, which is skewed towards the northwest. The skewing likely occurs for two reasons. First, flow from the northwest experiences greater resistance, thereby developing larger drawdowns. Second, the recharge from the trenches is more likely to be felt inside than outside of the wall. Nevertheless, the former effect may have been partly ameliorated by placement of the pumping well close to the cutoff wall so that the (ideally) radial flow paths would be parallel to the wall.

4.1.5 Analysis

Hantush (1967) presented closed-form analytical solutions for the drawdowns in two aquifers, separated by a semi-pervious layer, due to pumping from one of the aquifers. This solution is inherently unsteady, because all of the pumpage must ultimately come from storage in the aquifers. Therefore, for large time, drawdowns will diverge. However, during the pumping test, all of the pumped water was recharged to the overburden via trenches 4, 5, and 6. This recharge provides the source of water allowing the system to reach steady-state, both in the field and, by superposition, in the analysis.

Figure 4-10 shows the idealized geometry for the pumping test analysis. The aquifers are assumed to be infinite, with homogeneous and isotropic (but distinct) transmissivities T_1 and T_2, and uniform storativities S_1 and S_2. (Here, as elsewhere, subscripts 1 and 2 will refer to the overburden and the bedrock, respectively). The aquitard is also infinite, with uniform thickness, b', and homogeneous vertical hydraulic conductivity, K'. Flow is assumed to be horizontal in the aquifers and

FIG. 4-8 MEASURED BEDROCK DRAWDOWN CONTOURS

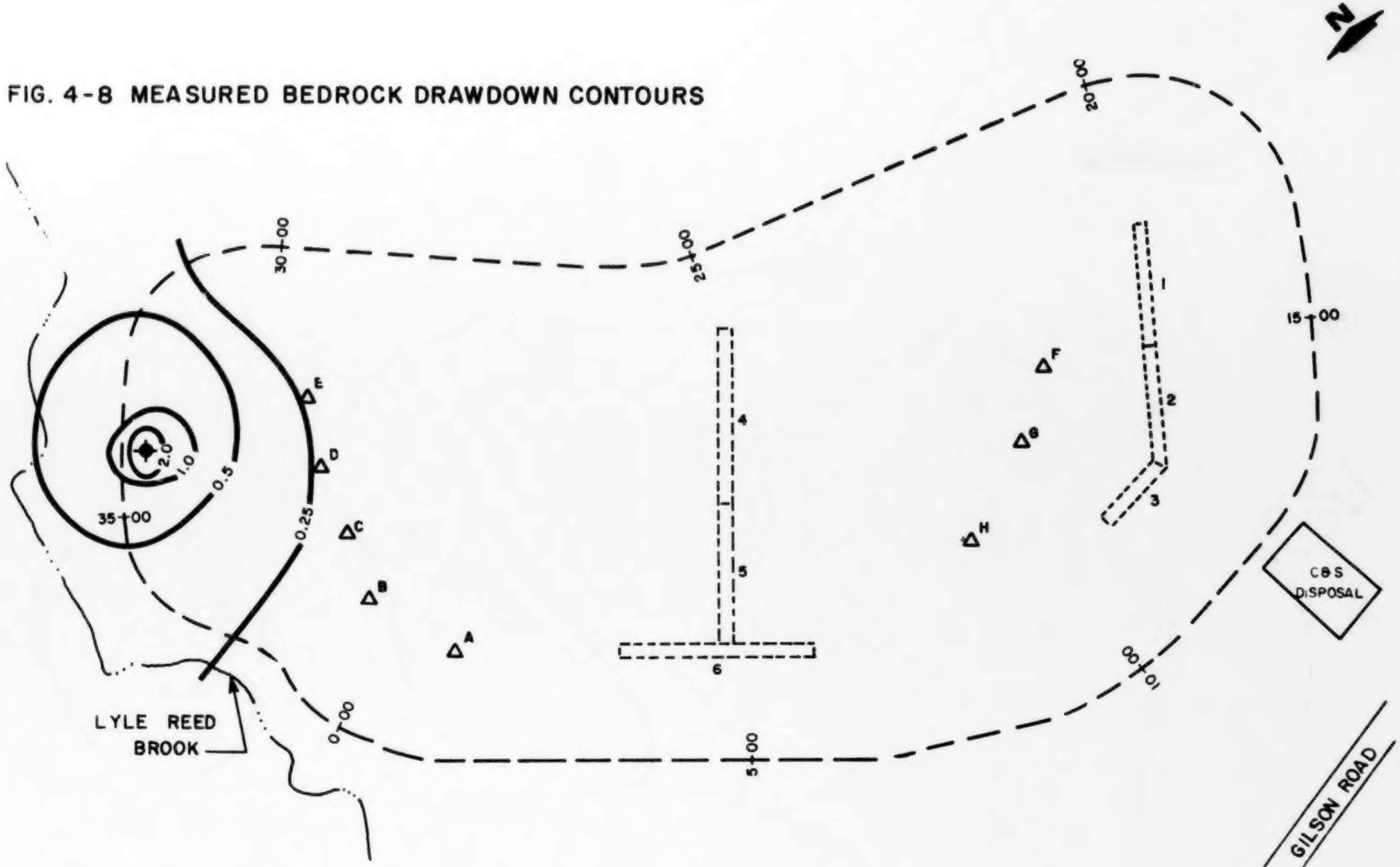

N
0+00
5+00
10+00
15+00
20+00
25+00
30+00
35+00
1
2
3
4
5
6
A
B
C
D
E
F
G
H
2.0
1.0
0.5
0.25
CBS DISPOSAL
GILSON ROAD
LYLE REED BROOK
0'
50'
100'
200'

FIG. 4-9 MEASURED OVERBURDEN DRAWDOWN CONTOURS

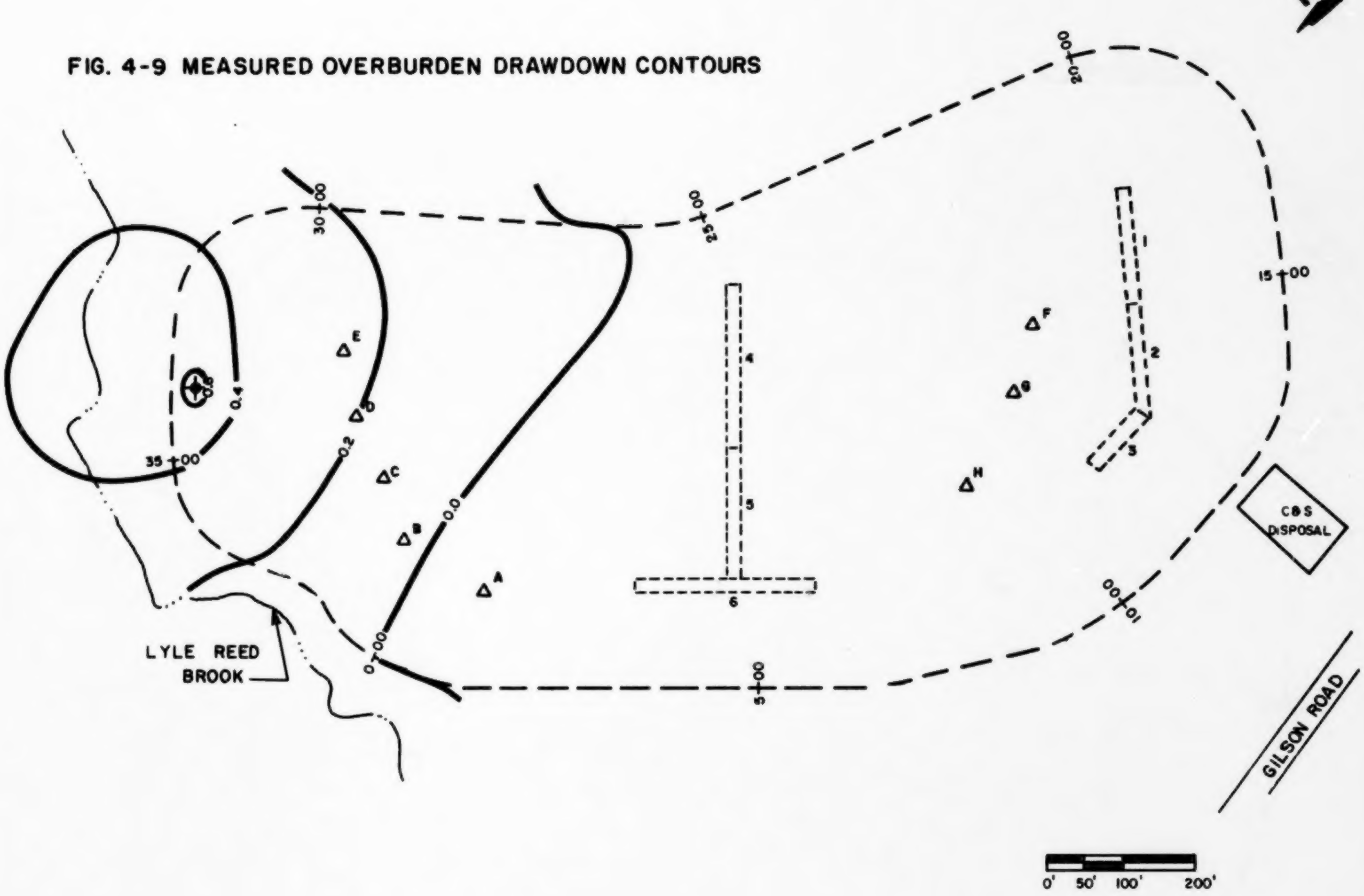
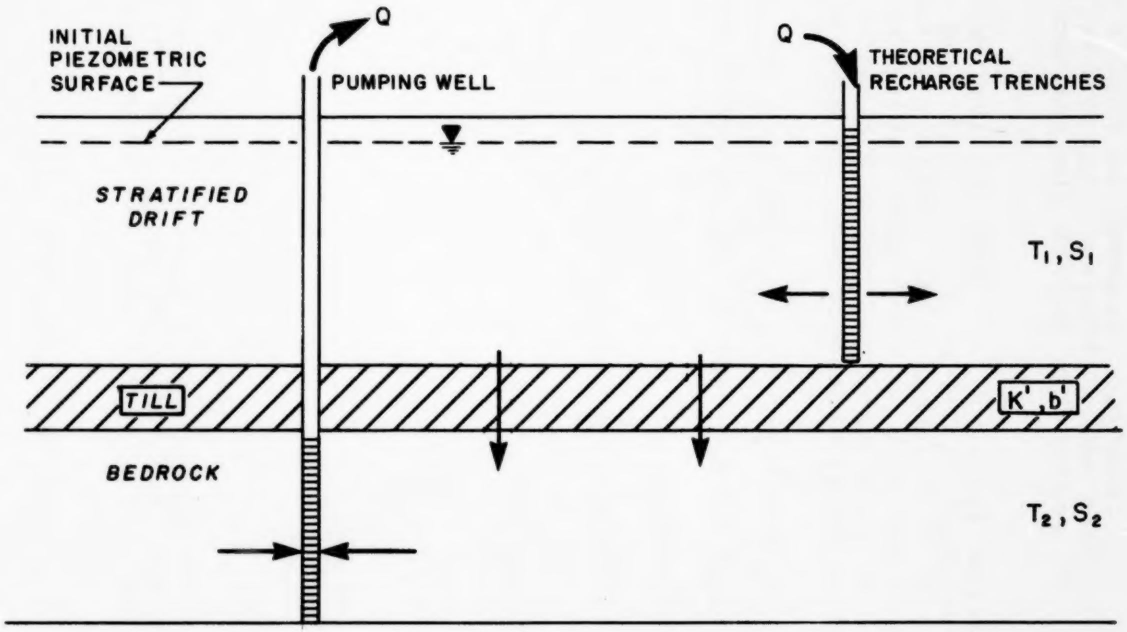

N
20+00
15+00
25+00
30+00
35+00
10+00
5+00
0+00
1
2
3
4
5
6
F
G
H
E
D
C
B
A
0.8
0.4
0.2
0.0
CBS DISPOSAL
GILSON ROAD
LYLE REED BROOK
0'
50'
100'
200'

FIG. 4-10 PUMPING TEST IDEALIZED GEOMETRY


INITIAL PIEZOMETRIC SURFACE
Q
PUMPING WELL
Q
THEORETICAL RECHARGE TRENCHES
STRATIFIED DRIFT
T_1, S_1
TILL
K', b'
BEDROCK
T_2, S_2

vertical in the aquitard. The amount of leakage through the aquitard is proportional to the difference in heads between the two aquifers. In addition, storage in the aquitard is neglected.

The actual elevations of the stratigraphic units vary considerably throughout the site. For example, the top of bedrock under the recharge trenches is higher than the top of till near the pumping well. However, approximating these units as horizontal is reasonable due to the semi-confining nature of the till.

The recharge trenches have been replaced mathematically by a single fully-penetrating (through the stratified drift) recharge well, as shown in Figure 4-10. While the recharge trenches actually add water only at the water table, approximating the trenches with a fully-penetrating well results in insignificant errors when calculating drawdowns near the pumping well.

Let s_{ij} be the drawdown in aquifer i due to pumping at a rate Q_j in aquifer j. Then, Hantush's solution for large time due to the pumping in the bedrock may be expressed as:

$$s_{12} = q_2 \left\{ \ln(\alpha r_2) + K_0(\beta r_2) \right\}$$

$$s_{22} = q_2 \left\{ \ln(\alpha r_2) - (T_1/T_2)\, K_0(\beta r_2) \right\}$$

$$q_2 = \frac{Q_2}{2\pi(T_1 + T_2)}$$

$$\alpha = \left(\frac{T_1 S_1 + T_2 S_2}{4.5\, t\, T_1 T_2} \right)^{1/2}$$

$$\beta = \left(\frac{T_1 + T_2}{T_1 T_2 \sigma} \right)^{1/2}$$

$$\sigma = \frac{b'}{K'}$$

where r_2 is the horizontal distance from the bedrock pumping well, t is the time since pumping began, and K(X) is the Modified Bessel Function of the second kind and order zero, of argument x. By symmetry,

$$s_{11} = q_1 \left\{ \ln(\alpha r_1) - (T_2/T_1)\, K_0(\beta r_1) \right\}$$

$$s_{12} = q_1 \left\{ \ln(\alpha r_1) + K_0(\beta r_1) \right\}$$

$$q_1 = \frac{Q_1}{2\pi(T_1 + T_2)}$$

where r_1 is the horizontal distance from the overburden well. The total drawdowns s_1 and s_2 are then given by

$$\begin{aligned} s_1 &= s_{11} + s_{12} \\ &= q\,\{\ln(r_2/r_1) + K_0(\beta r_2) + (T_2/T_1)\,K_0(\beta r_1)\,\} \\ s_2 &= s_{21} + s_{22} \\ &= q\,\{\ln(r_2/r_1) - (T_1/T_2)\,K_0(\beta r_2) - K_0(\beta r_1)\,\} \\ q &= q_1 + q_2 \end{aligned}$$

These solutions were evaluated numerically and contoured for varying values of the parameters T_1, T_2, and , commonly known as the leakance of the aquitard. This procedure was iterative. For example, if the calculated drawdowns in the bedrock were too large, then either the transmissivity of the bedrock was increased, or the leakance of the till was increased, or both. The values of these parameters which produced analytical contours providing the best fit to the measured drawdown contours (Figures 4-8 and 4-9) are:

- overburden transmissivity = 3,500 ft^2/d
- bedrock transmissivity = 6,500 ft^2/d
- glacial till leakance = 0.33 d^{-1}

The analytical drawdown contours calculated using these parameters are shown in Figures 4-11 and 4-12 for the fractured rock layer and the overburden, respectively. There appears to be good correspondence between the measured and computed drawdowns in the bedrock. The overburden match is less accurate. However, consideration must be accorded the heterogeneous nature of the geologic setting (especially the existence of the cutoff wall), the surface water boundaries nearby, and the highly idealized geometrical assumptions which were made.

The parameters thus calculated in this analysis are effective homogeneous values. As such, the analytical model can only represent a kind of mean behavior of the system. In particular, the irregularities in the overburden must be neglected.

FIG. 4-11 MODELED BEDROCK DRAWDOWN CONTOURS

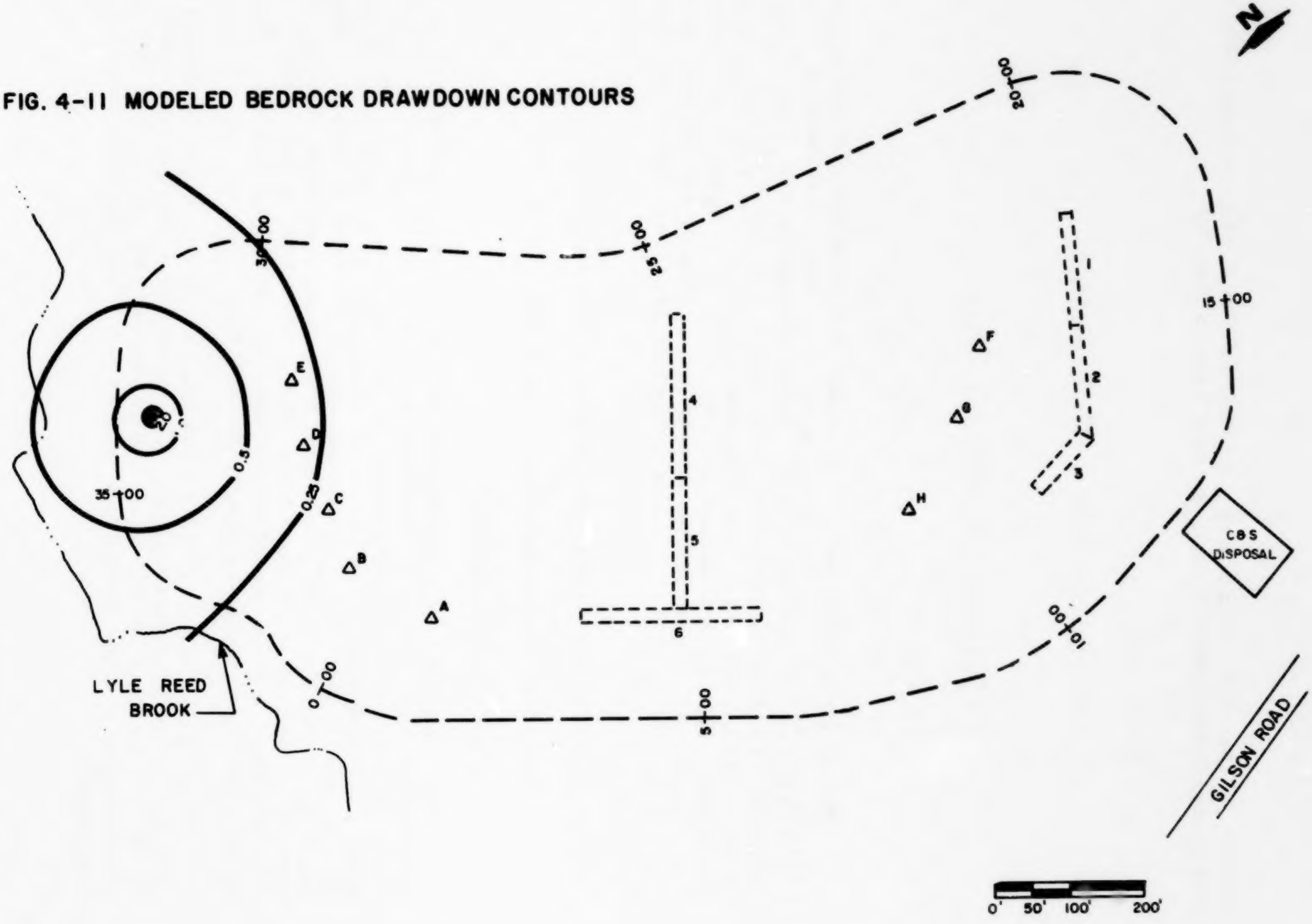
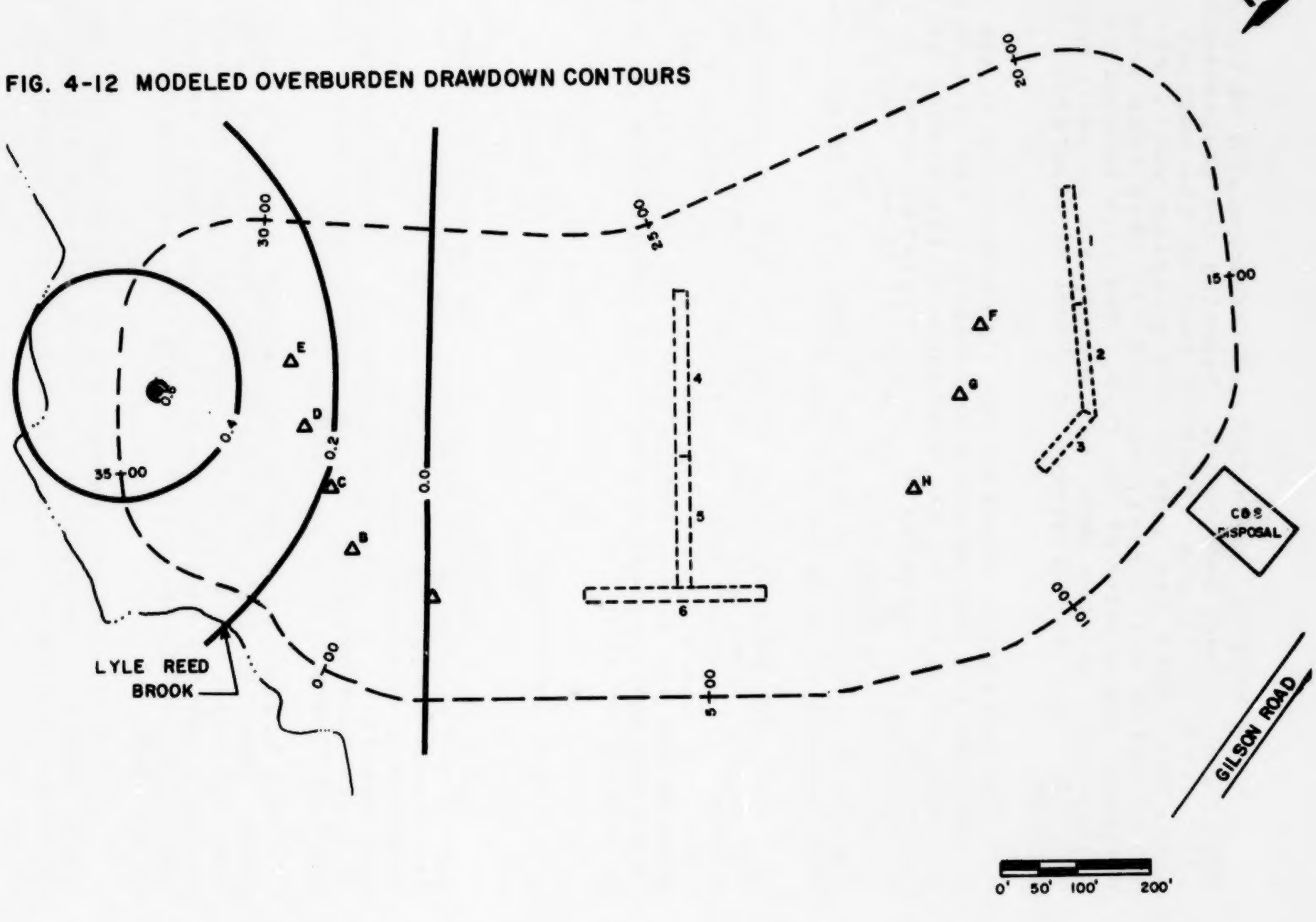

N
20+00
15+00
25+00
30+00
35+00
0+00
5+00
10+00
1
2
3
4
5
6
F
G
H
E
D
C
B
A
0.5
0.25
CBS DISPOSAL
GILSON ROAD
LYLE REED BROOK
0'
50'
100'
200'

FIG. 4-12 MODELED OVERBURDEN DRAWDOWN CONTOURS


N
20+00
15+00
25+00
30+00
35+00
10+00
5+00
0+00
0.6
0.4
0.2
0.0
E
D
C
B
F
G
H
1
2
3
4
5
6
CBS DISPOSAL
GILSON ROAD
LYLE REED BROOK
0'
50'
100'
200'

In the bedrock, the calculated transmissivity is higher than had previously been recognized. Assuming the transmissivity is concentrated in the upper 10 feet of the bedrock, as is consistent with the logs of the pumping well, PT-1, and observation wells PT-2, -3, and -4, then this zone has a hydraulic conductivity of 650 ft/d. The high bedrock hydraulic conductivity and its observed isotropy are mutually consistent and agree with the observed high level of fracturing in that stratum.

The effective homogeneous transmissivity (3,500 ft^2/d) calculated for the overburden, however, has probably been strongly affected by the presence of the cutoff wall, and therefore, is considered to be somewhat lower than the transmissivity of the native material alone. Therefore, the calculated transmissivity is considered to be useful only for lumping the behavior of the overburden as it affects the other stratigraphic units in the analysis.

Near the pumping well, the glacial till is approximately 15 feet thick. Given this thickness and the computed leakance, the glacial till would have an effective homogeneous vertical hydraulic conductivity of 5.0 ft/d. This is reasonable considering the discontinuous nature of this stratum, and the relatively small percentage of fines found in the glacial till underlying the site.

4.1.6 Conclusions

Based on a comparison between the measured drawdowns in the bedrock and stratified drift during a pumping test and analytical drawdowns similarly calculated using a mathematical model, effective homogeneous transmissivities have been evaluated for both aquifers. An effective homogeneous leakance for the glacial till aquitard was also derived. These calculated parameters compared favorably with descriptive field data.

The transmissivity of the bedrock was found to be 6,500 ft^2/d. Based on data obtained during drilling, it is apparent that the upper ten feet of the bedrock is highly fractured and is probably responsible for the bulk of the transmissivity. The hydraulic conductivity of this layer is therefore calculated to be 650 ft/d.

The calculated transmissivity of the overburden was 3,500 ft^2/d. However, the existence of the cutoff wall has undoubtedly affected this calculation. Therefore, this transmissivity is useful only for lumping the behavior of the stratified drift as it affects the other stratigraphic units.

The leakance of the glacial till was estimated to be 0.33 d^{-1}, a value which has probably been affected by the discontinuous nature of this stratum. Nevertheless, if the till is considered to be fifteen feet thick (on average), then the vertical hydraulic conductivity of the till can be estimated to be 5 ft/d.

4.2 Cutoff Wall Hydraulic Efficiency

The objective of this portion of the work was to evaluate the hydraulic efficiency of the cutoff wall. Cutoff efficiency is dependent upon the wall's bulk, as opposed to intact, hydraulic conductivity. The intact hydraulic conductivity represents the in-place backfill material alone, whereas the bulk hydraulic conductivity incorporates the combined effects of the intact material and any possible cracks and/or windows in the backfill and inadequacies of the key to bedrock. It is the bulk hydraulic conductivity which determines the large-scale hydraulic behavior of the wall.

The cutoff wall bulk hydraulic conductivity can only be evaluated indirectly, by hydraulically stressing the upper aquifer and cutoff wall while observing the hydrologic response. Ideally, the hydrologic response would be directly related only to cutoff wall hydraulic conductivity. At the Gilson Road site, however,both the cutoff wall in the upper aquifer and the underlying bedrock aquifer affect the flow of groundwater. In addition, the glacial till aquitard separating the two aquifers is relatively permeable, rendering it impossible to stress one aquifer without stressing the other. Therefore, it was necessary to separate multi-aquifer effects upon the groundwater system before the performance of the cutoff wall could be assessed. Data yielded by the bedrock pumping test (Section 4.1) provided a good estimate of the bulk, large-scale transmissivity of the bedrock aquifer and the leakance of the glacial aquitard. Thus, the effects of these units could be accounted for in the overall response of the upper aquifer.

A three-dimensional finite-difference flow model of the site had previously been calibrated for the design of the hydrodynamic isolation system. In addition, numerous multilevel monitoring installations existed at the site, both inside and outside of the cutoff wall. Therefore, cutoff wall bulk hydraulic conductivity could be evaluated by artificially increasing the cutoff hydraulic conductivity in the numerical model and performing a sensitivity analysis comparing simulated vertical piezometric head distributions to the actual field heads. The actual bulk hydraulic conductivity of the cutoff wall was taken to fall within the range of conductivities for which the computed and measured heads matched.

Based upon the above defined range of bulk conductivities, it was possible to assess the efficiency of the cutoff wall. Cutoff efficiency was defined as the ratio of flux through the overburden which had been cut off to that which would have occurred without the wall. These fluxes were also determined using the model.

4.2.1 Test Constraints

The investigation of post-construction cutoff wall efficiency should encompass the entire cutoff wall. The impact of design/construction issues such as varying wall depth, overburden type, magnitude and direction of bedrock slope relative to backfilling direction, etc. could then be addressed. However, the large effort associated with this work required a limited scope. As such, the effort concentrated on a 1,000 foot section of the downgradient wall. This portion of the wall was judged likely to provide the most salient data with respect to containment performance for a number of reasons. First, the large depth of the wall in this location (up to 105 feet) increased the potential for imperfections and thus for bulk conductivities exceeding specification limits. Second, under non-pumping conditions, this area exhibited the greatest head loss across the wall and thus the highest gradients to drive leakage through windows, if they existed. Third, the highest contaminant concentrations were found in the downgradient portion of the site. As such, a given degree of leakage in this area would result in the most significant environmental impact. Fourth, the greatest degree of groundwater manipulation was to be implemented in this area via the hydrodynamic isolation system. Additional advantages associated with investigation of the downgradient wall from a programmatic standpoint were previously presented in Section 4.1.1.

To evaluate cutoff wall bulk hydraulic conductivity via sensitivity analysis, the cutoff wall must be hydraulically stressed such that the variation in piezometric head in the vicinity of the cutoff is large in comparison to the calibration accuracy of the numerical model. To create this condition, the head difference across the cutoff wall must exceed two feet. Initially, it was envisioned that the 5 pumping wells associated with the existing hydrodynamic isolation system (see Figure 4-1) could be over-pumped to stress the downgradient wall. However, the large bedrock transmissivity, as documented by the bedrock pumping test (Section 4.1), precluded the development of adequate drawdown in the vicinity of the wall.

The criterion that testing should not cause off-site contaminant migration eliminated the possibility of increasing the internal heads as compared to the heads outside the wall. This could have been accomplished by shutting down the hydrodynamic isolation system and allowing the groundwater to mound against the inside face of the downgradient wall. If necessary, the head difference could have been further augmented by dewatering Lyle Reed Brook.

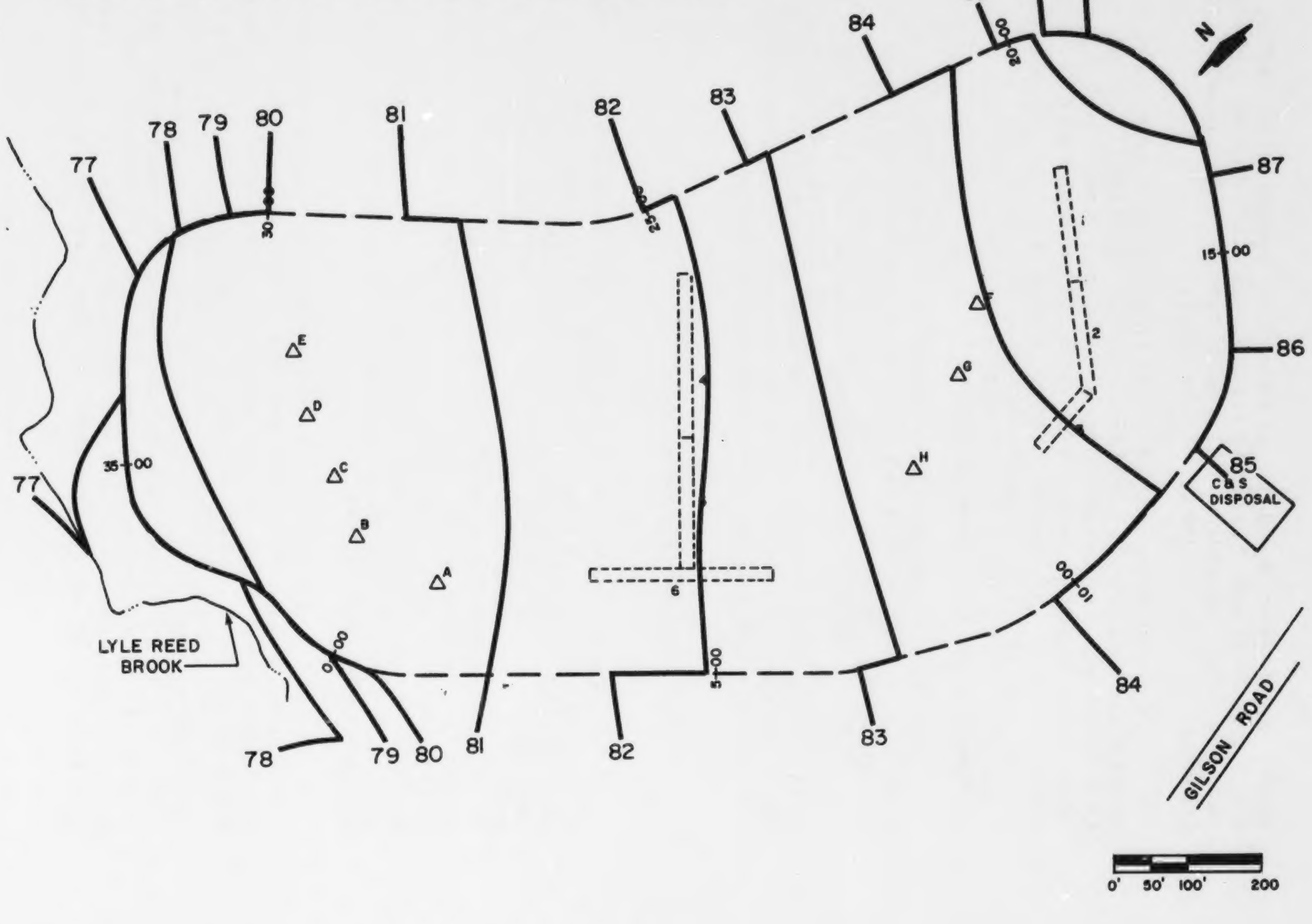
The above constraints eliminated potential mechanisms which could significantly stress the downgradient cutoff wall. Fortunately, previous water level monitoring data included rounds conducted after cutoff wall construction but prior to implementation of the hydrodynamic isolation system. One such data set, taken in June 1983, showed head differences across the wall of over 3 feet. This data set was therefore adopted as the reference data set for the sensitivity analysis.

4.2.2 Reference Data Set

The June 1983 reference data set includes piezometric heads obtained at 117 points in 56 multi-level and single level monitoring installations. The observation wells and piezometers used were installed according to the methods outlined in Section 4.1.3.1 In addition, water levels were taken at 16 surface stations along Lyle Reed Brook. This data set predates the bedrock pumping test, and as such, the monitoring installations installed for that test were not in place. However, there were enough multi-level installations to adequately define the vertical piezometric head distribution inside and outside of the downgradient cutoff wall.

Figure 4-13 shows contours of water table elevations during the June, 1983 reference period. The piezometric heads were measured over a three-day period, from June 3 to June 5, 1983. This data was taken after a very wet spring season, and represents the highest internal/external head differences measured at the site since cutoff wall construction. At the time the data was taken, the cutoff wall had just been installed and the recirculation system was not yet in operation. Near the eastern (upgradient) end of the containment, the heads outside the wall built up in response to the low hydraulic conductivity

FIG. 4-13 MEASURED WATER TABLE CONTOURS, JUNE, 1983

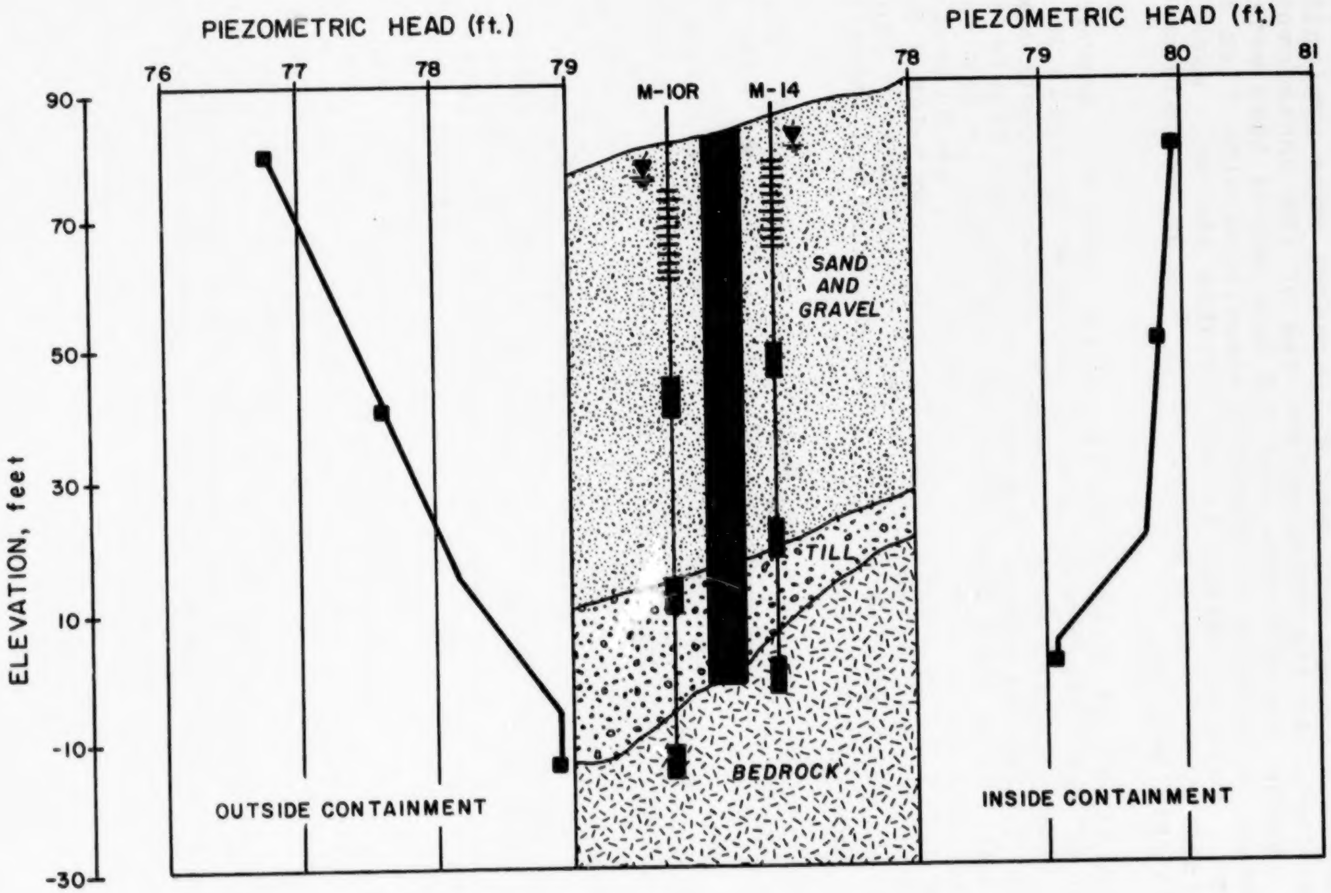

N
77
78
79
80
81
82
83
84
85
86
87
30
25+00
20+00
15+00
10+00
5+00
0+00
35+00
A
B
C
D
E
F
G
H
1
2
6
C & S DISPOSAL
GILSON ROAD
LYLE REED BROOK
0'
50'
100'
200

of the barrier. The total drop across the upgradient wall was approximately 2.5 feet. Along the north and south sides, flow was inward, although the head differences were typically less than one foot. At the downgradient end of the containment, the levels inside the wall were over 3.0 feet higher than outside the containment in response to mounding associated with flux through the site. Here the water levels outside the wall were largely determined by Lyle Reed Brook, which has an average slope of 0.002 in this section.

The vertical head distributions near the downgradient portion of the cutoff wall are illustrated by wells M-14 and M-10R. Installation M-14 is located inside the containment and M-10R is located outside of the containment (see Figure 4-1 for location). The head distributions are shown in Figure 4-14. Note that due to an error in seal placement, as discussed in Section 4.1.4.2, the data points for M-14-3 and M-10R-3 were not used. However, the shapes of the head distributions presented reflect a compilation of data from other multilevel installations adjacent to this location.

A number of trends in this data should be noted. First, in the upper aquifer, the data indicates a head drop of over 3 feet across the wall. The mounding on the inside of the wall reflects the impedance generated by the wall to flux through the containment. Second, on the inside of the wall, piezometric heads decrease with depth indicating downward flow. The converse is true outside of the wall. Note that if both the wall and the bottom aquiclude were totally impervious (no flow), the head distribution would be vertical on both sides of the wall (no head loss with depth). Third, the vertical gradient is greater in the glacial till than in the sand and gravel overburden as would be expected based on the disparity in hydraulic conductivity of the deposits. Finally, the head in the bedrock inside the containment is nearly identical to that outside the containment. This is consistent with a bedrock of high hydraulic conductivity (low head loss with flow) confined by a lower hydraulic conductivity glacial till, as defined during the bedrock pumping test (Section 4.1). The above data indicates that flux from upgradient in the containment mounds when it encounters the low hydraulic conductivity cutoff wall, the mounding drives flow through the glacial till aquitard, and the flow out of the containment is then concentrated in the fractured rock below the wall.

FIG. 4-14 VERTICAL HEAD DISTRIBUTIONS AT M-10R & M-14, JUNE, 1983



4.2.3 Numerical Methodology

The complexity of the hydrologic system at the Gilson Road site renders closed-form solutions unsuitable for analysis of cutoff bulk hydraulic conductivity. This is particularly true because the distribution of piezometric heads reflects the geologic heterogeneity in both the vertical and horizontal planes. In this case, over 1,000 lineal feet of cutoff wall was to be investigated. Therefore, a three-dimensional numerical model was required to correlate the piezometric head data generated when stress testing the wall. Such a model had previously been calibrated to the site for design of the hydrodynamic isolation system and thus, this capability was readily available.

As previously discussed, the Gilson Road site represents a coupled system where response to hydraulic stress is a function of not only the cutoff wall bulk hydraulic conductivity, but also the characteristics of the glacial till aquitard and the underlying bedrock aquifer. The first step in isolating the behavior of the cutoff wall from the rest of the hydrologic system was to better define the transmissivity and leakance of the bedrock aquifer and glacial till aquitard, respectively. This was accomplished via a bedrock pumping test as outlined in Section 4.1. The hydrologic parameters for the upper sand and gravel aquifer were known from pumping tests executed as part of the Remedial Investigation/Feasibility Study. Therefore, the only remaining hydrologic unit for which the parameters were unknown was the cutoff wall itself.

Under hydrologic stress conditions, the horizontal vs. vertical flux through the upper aquifer is a function of the unknown bulk hydraulic conductivity of the cutoff wall as compared to the known conductivities of the underlying aquiclude/aquifer system. The distribution of vertical piezometric heads reflects the flux vector. For high wall conductivities, relative to conductivities below the wall, the flux would primarily be horizontal through the upper aquifer, yielding little variation in head with depth. For low relative wall hydraulic conductivities, flux would be more vertical commensurate with flow under the wall. This condition would be reflected in varying piezometric head with depth. Naturally, the degree of variation in the vertical heads is also dependent on the relative conductivities of the units adjacent to and below the wall. The multi-level monitoring network established at the site allowed measurement of the vertical head distribution for various stress states. This data, in conjunction with the numerical model, provided the capability for cutoff wall bulk hydraulic conductivity evaluation via numerical sensitivity analysis.

A range of bulk conductivities were input into the calibrated model and the vertical heads generated were compared to the actual field head distribution for a given stress state, the June 1983 scenario. Bulk conductivities which produced heads matching the field data, to within the model calibration accuracy, were taken to be possible representations of the actual state of the cutoff wall. Conversely, simulated bulk conductivities which did not generate matching heads were assumed to be invalid.

4.2.3.1 Model Description

The sensitivity analysis was performed using a three-dimensional finite-difference program (described in Trescott (1975) and Torak (1982)). Features of this code include aquifer heterogeneity and anisotropy, steady or transient simulations, variable grid spacing and a phreatic surface in the uppermost layer. The solution is found using the strongly implicit procedure.[3]

A cutaway view of the model is shown in Figure 4-15. A plan view of the top layer is shown in Figure 4-16, with the nodes representing the cutoff wall, Lyle Reed Brook, and Trout Brook indicated. The model has five layers vertically, and a 48 x 50 grid horizontally. The horizontal grid spacing varies from 180 feet near the boundaries to 40 feet in the vicinity of the cutoff wall. The bottom layer of the model is 50 feet thick, with its center at elevation -28 feet (with respect to the City of Nashua datum). The next layers are 30, 20, 20, and 50 feet thick, respectively. The top layer is artificially thick to allow the water table to vary without restraint. This extra thickness is not shown in Figure 4-15.

Most of the borders of the model, including those representing constant heads, were no-flow boundaries. The one exception is at the downgradient end of the model, where a fixed constant flux was simulated by a series of virtual pumping wells. Precipitation-generated recharge was applied uniformly as a source to the nodes in the top layer, except in areas covered by the impermeable surface cap. The recharge which would have been applied in these areas was added to nodes around the cap, in accordance with the known cap contours.

---

[3] A numerical technique used to solve the piezometric head matrix.

FIG. 4-15 CUTAWAY VIEW OF NUMERICAL MODEL

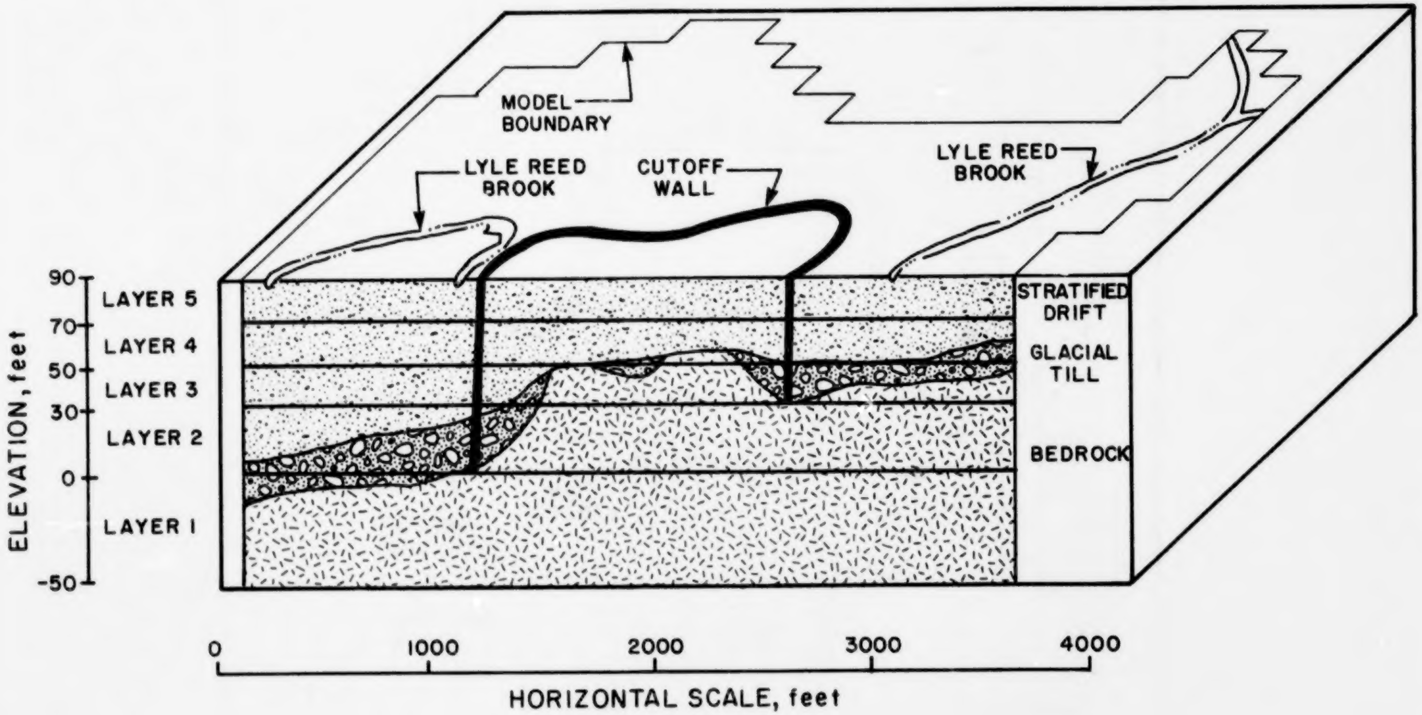
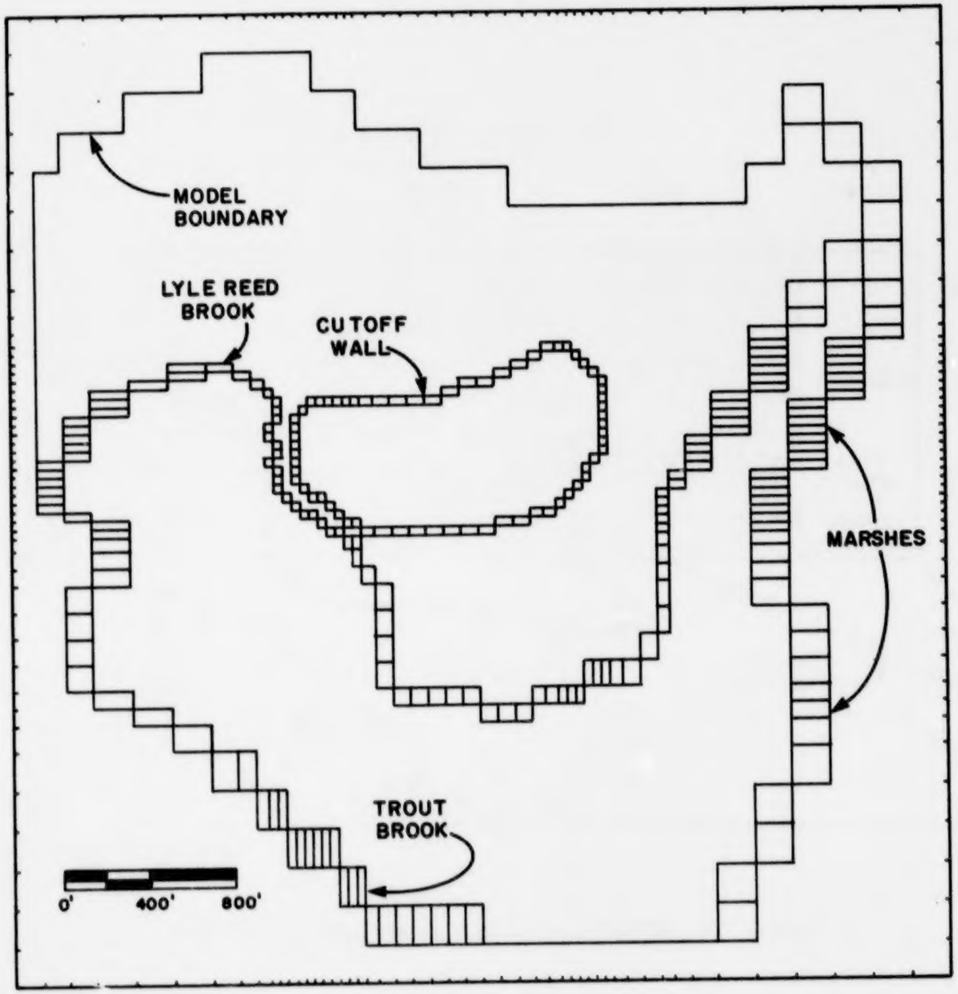

MODEL BOUNDARY
LYLE REED BROOK
CUTOFF WALL
LYLE REED BROOK
90
70
50
30
0
-50
ELEVATION, feet
LAYER 5
LAYER 4
LAYER 3
LAYER 2
LAYER 1
STRATIFIED DRIFT
GLACIAL TILL
BEDROCK
0
1000
2000
3000
4000
HORIZONTAL SCALE, feet

FIG. 4-16 PLAN VIEW OF TOP LAYER OF MODEL


MODEL BOUNDARY
LYLE REED BROOK
CUTOFF WALL
MARSHES
TROUT BROOK
0'
400'
800'

During the calibration procedure, the cutoff wall hydraulic conductivity simulated was the design hydraulic conductivity of 1 x 10^{-7} cm/sec, as based on quality control test data (Section 2). The actual conductivities of the model nodes representing the wall were, however, ten times greater in order to account for the 40 foot node width as compared to the 3 to 4 foot wall thickness.

In order to reduce complexity, the model was used to simulate steady-state conditions only. No transient simulations were performed for this effort. Each simulation treated all of its boundary conditions as constant, although the values of the boundary parameters were changed from simulation to simulation. In cases where a condition was actually time-varying, it was replaced by an equivalent steady-state condition. The most obvious example of this is the areal recharge due to precipitation. For example, in the calibration for the June, 1983, scenario, the applied recharge was 42 inches/year. This value exceeds the probable yearly average recharge to the area by a factor of two. However, it is reasonable as an effective steady-state recharge for the three-month period preceding that date, as based on meteorological data.

The other transient conditions simulated by the model were judged to be changing slowly enough that their values at any point in time could be (and were) used as steady-state input. For example, constant head nodes were used to represent Lyle Reed Brook and Trout Brook, as well as the swamps and marshes upgradient of the site. The specified heads varied spatially and between simulations, but were constant at any one node during any one simulation.

4.2.3.2 Model Calibration

The model had originally been calibrated for two water level data sets, representing the system's state in January, 1982, and June, 1983. The current effort included further calibration to reproduce two additional water level data sets, (October 1983 and September 1985), for a total of four different stress conditions. (The four data sets will be abbreviated as JAN82, JUN83, OCT83, and SEP85). The stress conditions which these scenarios represent are shown in Table 4-2. The additional calibration effort incorporated improved understandings of system behavior as well as new, better and/or more extensive data. In particular, the bedrock transmissivity which was determined from the bedrock pumping test was explicitly incorporated. Also, further data regarding seasonal variations in surface water levels proved helpful in improving the ability of the model to represent the behavior of the aquifer/cutoff wall system in response to different recharge rates.

Parameters varied during model calibration included the hydraulic conductivities and anisotropies of the soils, the amount and distribution of recharge, and the amount of subsurface drainage (constant flux) out of the model at the downgradient end. The values of recharge and constant flux used in each of the calibrated simulations are shown in Table 4-2. In all cases, parameter variation remained within ranges previously established based on the subsurface data amassed for the site.

TABLE 4-2

STRESS SCENARIOS AND SIMULATION FLUXES

Date	Wall & Cap	H.I.[1] Sys. On	Constant[2] Flux (cfs)	Recharge (in/yr)	Recharge Descriptive
JAN82	no	no	0.240	20	medium
JUN83	yes	no	0.363	42	high
OCT83	yes	no	0.128	6	low
SEP85	yes	yes	0.240	8	low

1 Hydrodynamic isolation system
2 Flux rate for downgradient boundary condition

Three measures of the degree of calibration were used. First, statistics were calculated for the deviations between measured piezometric heads and those calculated at the homologous nodes in the model. This provided a quantitative estimator of the degree of calibration as a whole. Second, visual comparison of water table elevation contours ensured that important qualitative features were preserved. Finally, care was exercised to see that the vertical distributions of modeled heads matched well with the field data.

The statistical measures used were the maximum and minimum deviations between measured and modeled heads, the average deviation, the average of the absolute deviation, and the RMS deviation. The latter measure is equal to the standard deviation when the average deviation is zero. These statistics are presented for each calibration run in Table 4.3 and indicate that all of the runs are calibrated to a high degree.

TABLE 4-3

CALIBRATION DEVIATION STATISTICS[1]

Date	Maximum	Minimum	Average	Ave. ABS	RMS
JAN82	0.84	-0.77	-0.08	0.27	0.35
JUN83	1.11	-0.70	0.06	0.33	0.41
OCT83	0.75	-0.87	0.04	0.30	0.36
SEP85	1.33	-0.64	0.17	0.29	0.43

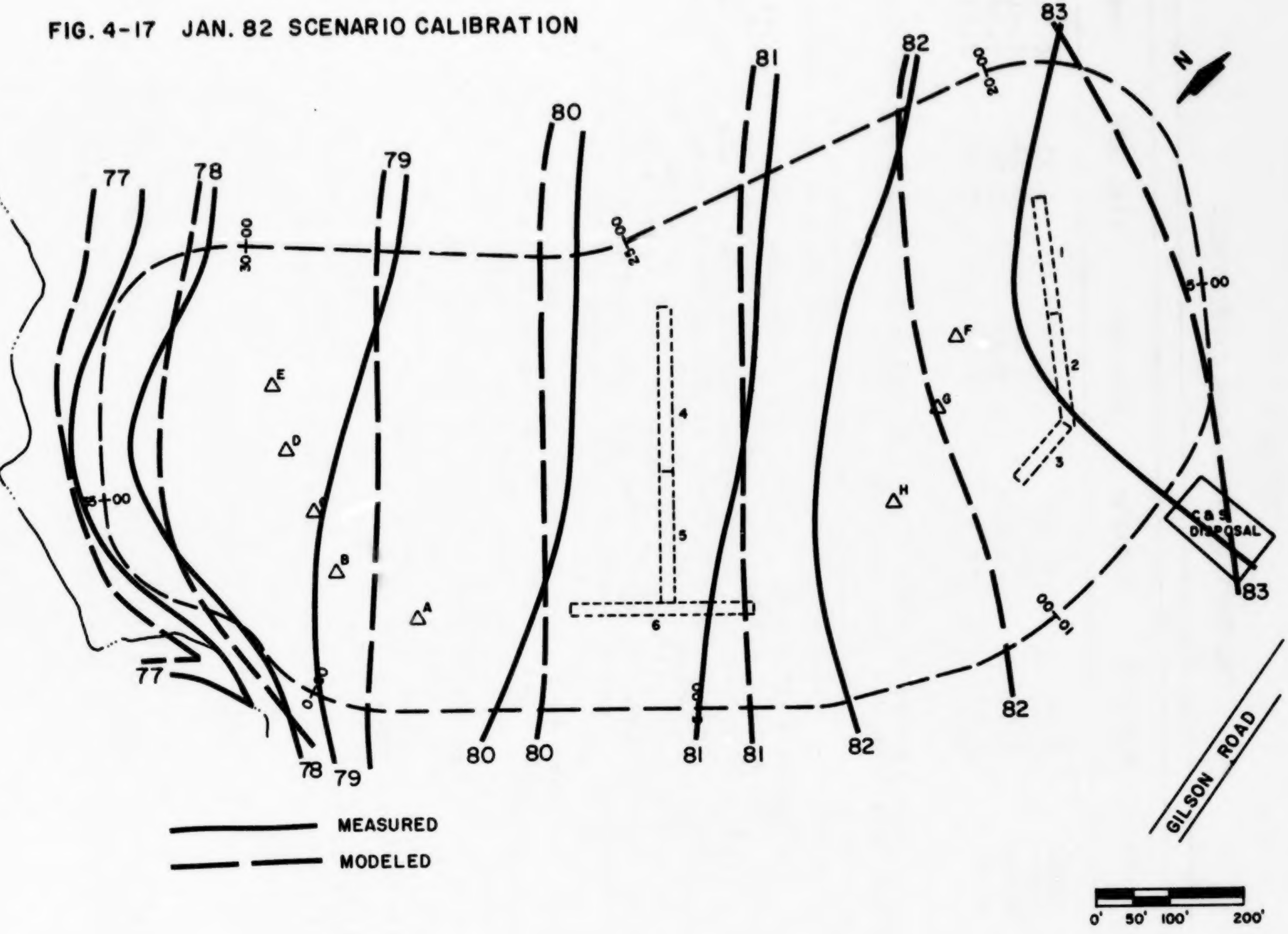
[1] All values in feet. Data analysis included the piezometric head data from all five layers.

Figures 4-17, 4-18, 4-19, and 4-20 show the measured water table contours for the four calibrated scenarios and their modeled counterparts. In general, the calibration was quite good as demonstrated by the figures, although there were some minor discrepancies which could not be resolved. In JAN82, most of the contours match, although the modeled heads were somewhat lower than the measured heads in the southern portion of the site. The OCT83 attempt however, suffers with respect to the contours from 78 to 80 feet. In SEP85, the field data shows a steep gradient from trenches 4, 5, and 6 towards pumping wells A through E, but then a very flat gradient from the wells to the wall. This abrupt change in slope was difficult to capture in the model, resulting in a high maximum deviation for the run. It is hypothesized that the mismatch was due to the effects of simulating the wells and trenches with the relatively coarse grid. Nevertheless, the discrepancy should not strongly affect leakage estimates because it is located away from the highly sensitive upgradient and downgradient ends of the site. Note that the modeled head differences across the downgradient portion of the wall are similar to those shown by the field data.

4.2.4 Sensitivity Analysis

The sensitivity analysis was performed by varying the cutoff wall bulk hydraulic conductivity (the parameter under investigation) in the model and evaluating the degree of correspondence between the numerically generated head

FIG. 4-17 JAN. 82 SCENARIO CALIBRATION



N
77
78
79
80
81
82
83
30+00
25+00
20+00
15+00
10+00
5+00
35+00
A
B
C
D
E
F
G
H
1
2
3
4
5
6
C & S DISPOSAL
GILSON ROAD
MEASURED
MODELED
0'
50'
100'
200'

FIG. 4-18 JUNE 83 SCENARIO CALIBRATION

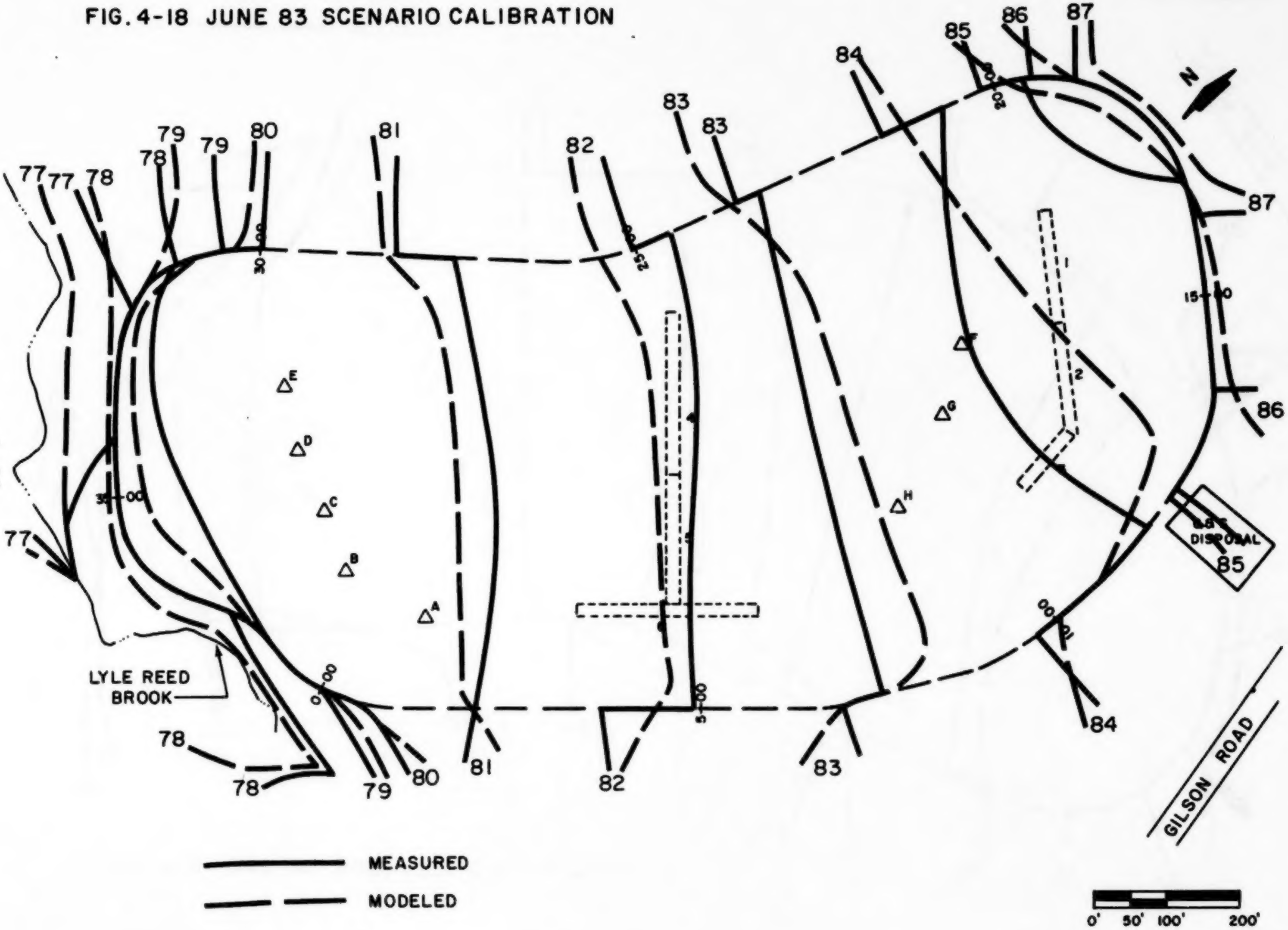

77
77
78
78
79
79
80
81
82
83
83
84
85
86
87
87
86
84
83
82
81
80
79
78
78
77
30+00
25+00
20+00
15+00
10+00
5+00
0+00
35+00
A
B
C
D
E
F
G
H
1
2
DISPOSAL
85
LYLE REED BROOK
GILSON ROAD
N
MEASURED
MODELED
0'
50'
100'
200'

FIG. 4-19 OCT. 83 SCENARIO CALIBRATION

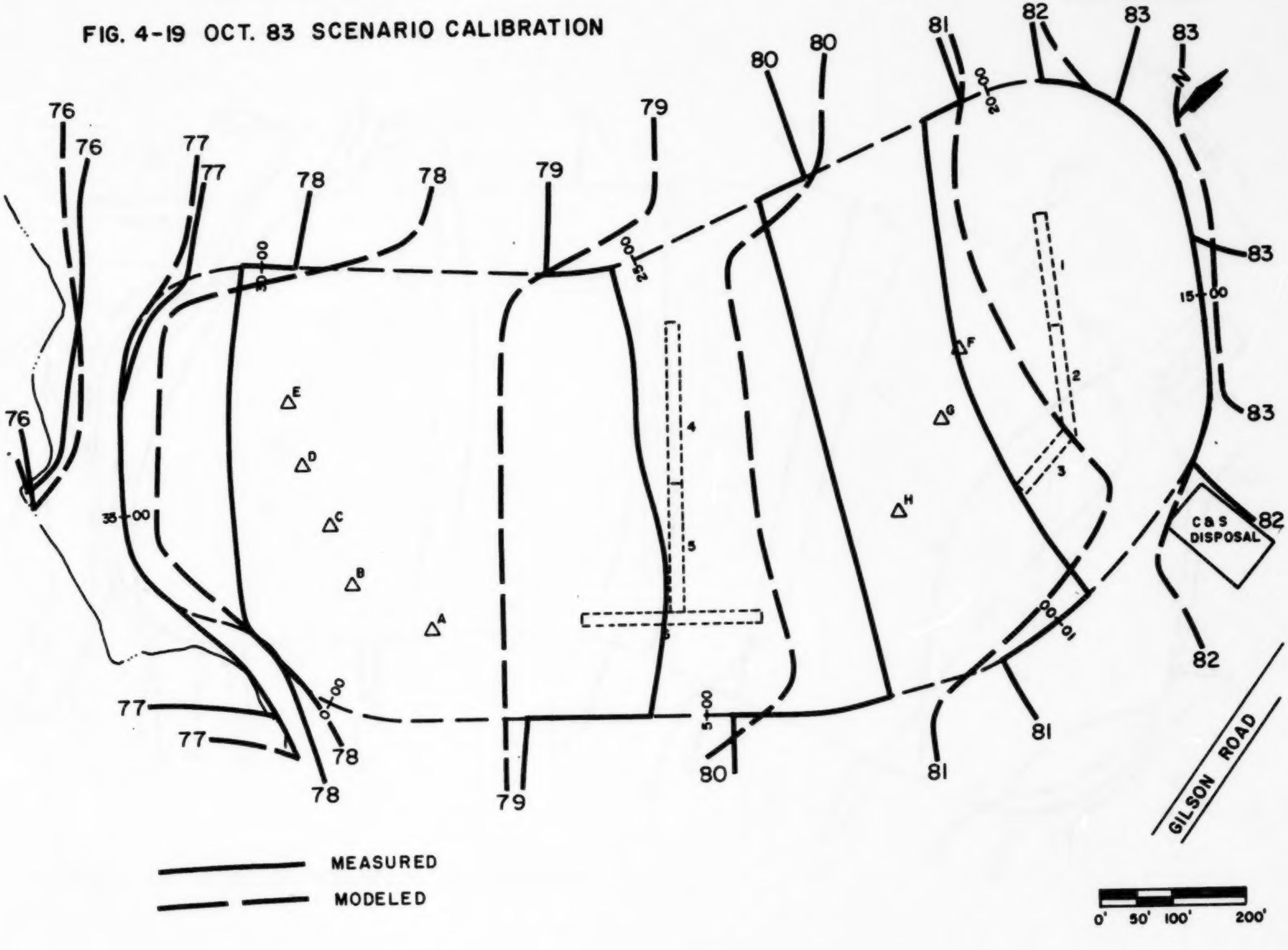
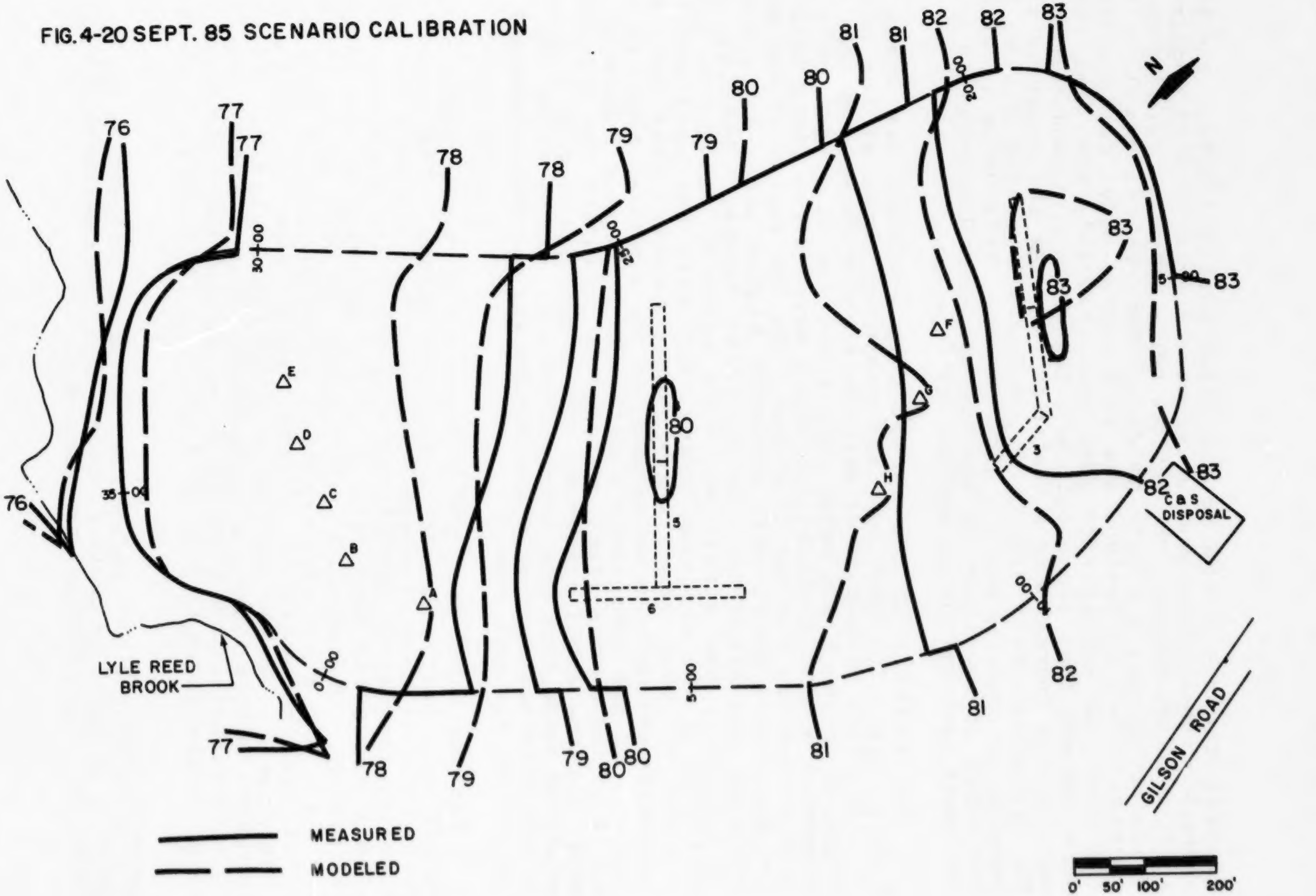

76
76
76
77
77
77
77
78
78
78
78
79
79
79
80
80
80
81
81
81
82
82
82
83
83
83
83
N
15+00
20+00
25+00
30+00
35+00
5+00
10+00
0+00
A
B
C
D
E
F
G
H
1
2
3
4
5
C&S DISPOSAL
GILSON ROAD
MEASURED
MODELED
0'
50'
100'
200'

FIG. 4-20 SEPT. 85 SCENARIO CALIBRATION


76
77
78
79
80
81
82
83
N
30+00
25+00
20+00
35+00
5+00
0+00
LYLE REED BROOK
C & S DISPOSAL
GILSON ROAD
A
B
C
D
E
F
G
H
1
3
5
6
MEASURED
MODELED
0'
50'
100'
200'

distributions and the actual field measured data. The bulk hydraulic conductivity was varied over four orders of magnitude in five equal steps. The initial value input for bulk hydraulic conductivity was 1 x 10^{-7} cm/sec. This value, equal to the specification criterion, was used during model calibration[4]. The next four sensitivity runs used precisely the same input data, except that the cutoff wall bulk hydraulic conductivity in each run was increased by a factor of ten over the previous run. The greatest bulk hydraulic conductivity used during the sensitivity analysis was 1 x 10^{-3} cm/sec. This value is within one order of magnitude of the average hydraulic conductivity of the upper sand and gravel aquifer (10^{-2} cm/sec).

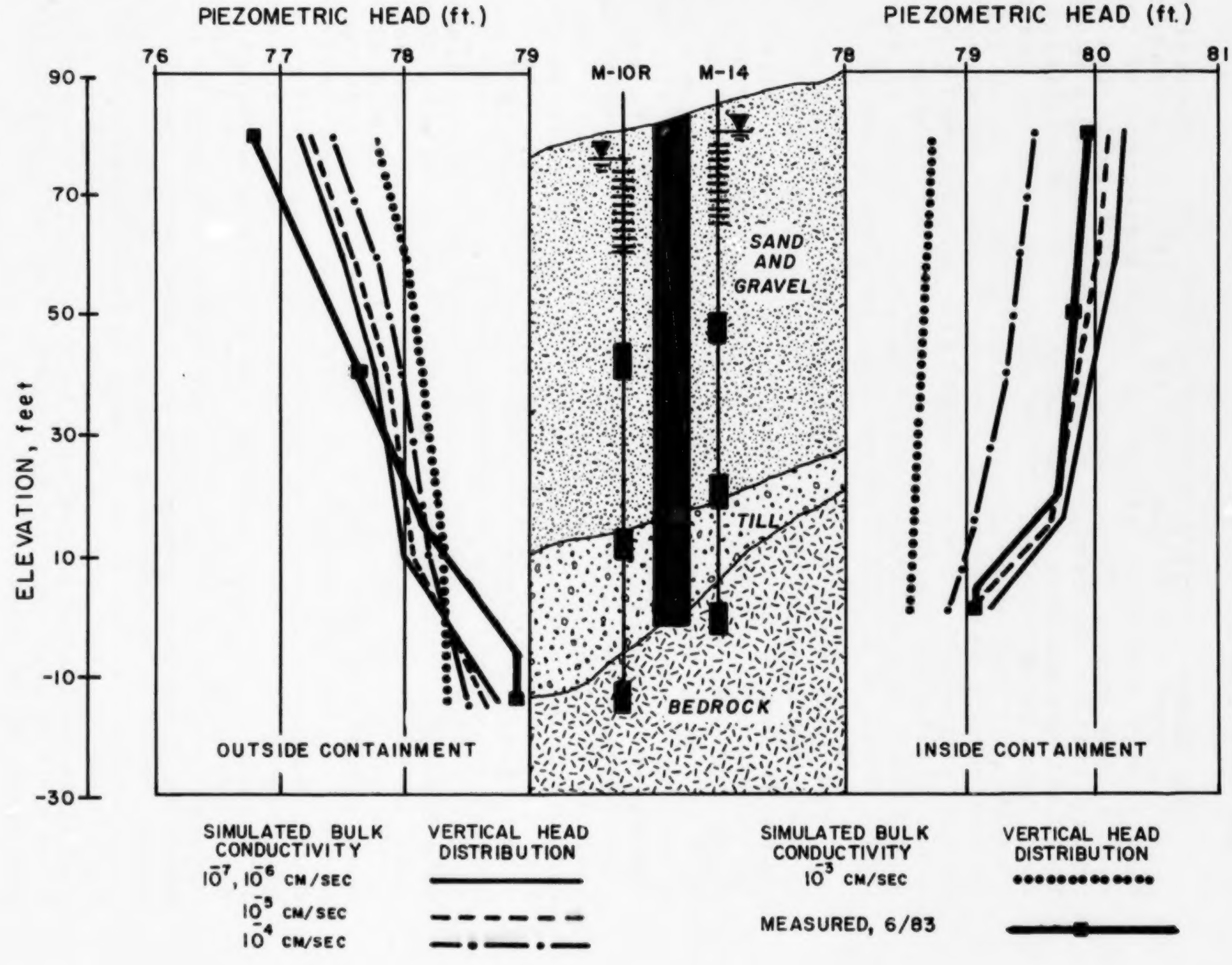
The results of the sensitivity analysis are shown in Figure 4-21. For each run, the computed heads at the nodes corresponding to multilevel wells M-14 and M-10R were plotted and compared to the field data. The first run (1 x 10^{-7} cm/sec) generated a head distribution which matched the field data to within about one quarter of a foot. On the inside of the wall the simulated heads were somewhat higher than, but closely followed the trend of, the measured heads. Outside the containment, the simulated heads again followed the trend exhibited by the measured heads, but were not consistently greater or less than the measured heads. This data set represents the calibration accuracy of the numerical model in the vicinity of the downgradient wall. As previously indicated, it was generated as the final run used to calibrate the numerical model for the JUN83 scenario. The second run (1 x 10^{-6} cm/sec) generated heads that were nearly identical to the first run. A single piezometric head distribution was therefore presented on Figure 4-21 for both runs.

For the third run, the bulk hydraulic conductivity of the cutoff wall was increased to 1 x 10^{-5} cm/sec. The generated heads in this case diverged noticeably from the first two runs. However, they still correlated with the field measurements to within the error band defined by the accuracy of the initial calibration. Therefore, 1 x 10^{-5} cm/sec, as well as 10x 1^{-6} and 1 x 10^{-7} cm/sec, represents a possible cutoff wall bulk hydraulic conductivity to the extent that it meets the constraints imposed by the sensitivity analysis. Further increases in bulk hydraulic conductivity (runs 4 and 5) resulted in head distributions which no longer matched the field data. As such, these values (1 x 10^{-4} and 1 x 10^{-3} cm/sec) do not

---

[4] The value of 1 x 10^{-7} cm/sec used for model calibration was selected as based on quality control field data obtained during cutoff wall construction; see Section 2.

FIG. 4-21 MODELED VERTICAL HEAD DISTRIBUTIONS FOR DIFFERENT WALL CONDUCTIVITIES



represent valid possibilities for the bulk hydraulic conductivity of the cutoff wall.

The above analysis indicates that bulk conductivities lower than 1 x 10^{-5} cm/sec are all, to the extent definable by the sensitivity analysis, plausible representations of the cutoff wall's impedance to flux. The analysis also indicates that bulk conductivities in excess of 1 x 10^{-5} cm/sec begin to yield head distributions which no longer match the field condition and are thus considered invalid. It is emphasized that the analysis only demonstrates that the value of 1 x 10^{-5} cm/sec is an upper bound on the cutoff wall hydraulic conductivity. This should not be misinterpreted to mean that the actual bulk hydraulic conductivity of the cutoff wall is 1 x 10^{-5} cm/sec. In fact, significant data indicates that the actual value is much lower, in the range of 1 x 10^{-7} cm/sec (see Sections 2 and 3).

The inability of the sensitivity analysis to provide a definitive evaluation of the cutoff wall bulk hydraulic conductivity, with respect to the specification criterion of 1 x 10^{-7} cm/sec, is a direct result of the much higher conductivities associated with the containment bottom (glacial till and bedrock). This is due to the coupled hydraulic behavior of the containment system which is analogous to an electrical circuit containing resistors in parallel. As such, the sensitivity analysis can only resolve the hydraulic conductivity of a specific individual hydraulic element to a degree proportional to the highest hydraulic conductivity exhibited by any element in the system. This is demonstrated by the data in Figure 4-21. As the cutoff wall hydraulic conductivity is increased from 10^{-7} cm/sec, the bedrock flux continues to dominate the heads, and thus the amount of fluid flux through the wall increases roughly linearly with it's hydraulic conductivity. The increase in flux through the wall is small enough, however, that it has no discernable effect on the heads. As the wall flux eventually becomes significant with respect to the bedrock flux, a concomitant decrease in the overall gradient occurs as reflected in the vertical head distribution. The high bedrock transmissivity, therefore effectively masks the cutoff wall hydraulic conductivity below 10^{-5} cm/sec. If the bedrock had a lower transmissivity, then this critical masking level would occur at a lower wall hydraulic conductivity. An upper bound could then be established which would be more consistent with the value specified for the cutoff wall bulk hydraulic conductivity (1 x 10^{-7} cm/sec). This limitation is inherent in any hydraulic stress analysis. Furthermore, in as much as hydraulic analysis is the only mechanism available to evaluate bulk hydraulic conductivity, the limitation applies to the viability of post-construction verification in general.

4.2.5 Cutoff Wall Efficiency

The effectiveness of the cutoff wall as a barrier technology is a function of its ability to arrest fluid flux. It is therefore necessary to determine the amount of flux before and after the installation of the wall. The effectiveness of the wall may then be quantified as the ratio of flux through the overburden which has been eliminated by the cutoff wall, to that which would have existed without the wall. This ratio is defined herein as the "wall efficiency".

An alternative definition of efficiency is the ratio of total flux through the entire site (overburden and rock) which has been cut off to total flux through the entire site before the installation of the wall. However, this ratio is suited only to an evaluation of the effectiveness of the entire passive containment system (cap, cutoff wall and bottom aquiclude) and not the effectiveness of the wall as a technology *per se*. This ratio is termed the "containment efficiency".

The flux values required to compute the above ratios were determined using a postprocessor adjunct to the flow model. In this program, the transmissivity and head matrixes are used to calculate the fluid flux out of the site through each node in or under the simulated cutoff wall. The resulting fluxes are reported by layer, by horizontal node, and by element type (wall or rock).

In addition to the three calibration runs[5] JAN82, JUN83, and OCT83, six more runs were performed on the model. These six runs represented the system under the combined conditions of low, medium, and high recharge, with and without the cutoff wall. A cutoff wall hydraulic conductivity of 1×10^{-7} cm/sec was used for the wall. The postprocessor was used to calculate the fluid fluxes through the rock and overburden/cutoff wall nodes in each of these runs. The results are shown in Table 4-4.

---

[5] The SEP85 run is not included herein because it models a scenario with the hydrodynamic isolation system in operation. As such, flux through the site/wall is a function of pumping rates as well as barrier conductivities.

TABLE 4-4

MODELED OUTWARD FLUXES[1]

Recharge	No Containment Wall[2]	No Containment Rock	Wall and Cap Wall[3]	Wall and Cap Rock
low	74,386	29,635	73	65,001
medium	89,248	32,833	79	66,578
high	121,982	39,116	93	73,668

[1] all values in gallons per day
[2] native overburden permeabilities
[3] wall hydraulic conductivity = 1×10^{-7} cm/sec

Table 4-4 shows that, before installation of the cutoff wall, the flux through the rock is about one-fourth of the total flux through the site. After cutoff installation, the rock flux is roughly doubled, due to the increased gradients created by the wall. However, the flux through the overburden is effectively eliminated. Therefore, the containment efficiency of the passive physical barriers at the Gilson Road site is approximately 55 percent under worst case precipitation induced stress conditions. Please note, however, that this does not include the effects of the hydrodynamic isolation system, (the active component of the total containment) which is designed to completely eliminate off-site migration of contaminated groundwater.

The sensitivity runs used to establish an estimate for cutoff wall bulk hydraulic conductivity were also passed through the postprocessor. The calculated fluxes out of the site for these runs are shown in Table 4-5. Based on these fluxes and the entry in Table 4-4 for high recharge[6] no containment, it is possible to calculate the wall efficiency for each run. Based on the sensitivity analysis, the cutoff wall bulk hydraulic conductivity was determined to be less then 1×10^{-5} cm/sec. Therefore, Table 4-5 shows that the wall efficiency must be greater than 93 percent.

---

[6] The high recharge data for the no containment case was selected to match the JUN83 scenario upon which the sensitivity analysis, and thus the data in Table 4-5, was based.

TABLE 4-5

SENSITIVITY RUN FLUXES

Cutoff Wall Bulk K (cm/sec)	Flux through the Site (gallons per day) Total	Wall	Rock	Wall Efficiency
10^{-7}	73,761	93	73,668	99.92%
10^{-6}	74,117	830	73,287	99.32%
10^{-5}	77,417	8,361	69,056	93.15%
10^{-4}	95,800	45,036	50,764	63.08%
10^{-3}	126,165	90,814	35,351	25.55%

4.2.6 Summary

The bulk hydraulic conductivity of the downgradient section of the Gilson Road cutoff wall was evaluated using a numerical sensitivity analysis. Three-dimensional finite-difference flow modeling provided comparison of simulated vertical piezometric head distributions to actual field measures head distributions for varying values of cutoff wall bulk hydraulic conductivity. An upper bound of 1 x 10^{-5} cm/sec was thus established for the cutoff wall. Further definition of the actual bulk hydraulic conductivity below this value was not possible using stress testing due to the large bedrock tramsmissivity and the practical limits on the calibration of the model. However, the considerable quality control data obtained during construction (Section 2) indicates that the hydraulic conductivity of the wall is approximately 1 x 10^{-7} cm/sec. This value is consistent with the stress testing results in that stress testing yielded an upper bound only.

Inasmuch as only an upper bound value for the cutoff wall bulk hydraulic conductivity could be determined via stress testing, evaluation of the wall efficiency (ratio of the overburden flux eliminated to the overburden flux without the containment) was limited to a lower bound solution. Based on the results of the sensitivity analysis, the wall efficiency for the cutoff wall was found to be greater than 93 percent. This is based solely upon the upper bound value for bulk hydraulic conductivity established by the sensitivity analysis, and as such is a worst case condition. A more probable value for wall efficiency is over 99% as obtained based on the quality control data which indicates a wall hydraulic conductivity of approximately 1×10^{-7} cm/sec.

4.3 Groundwater Quality Monitoring

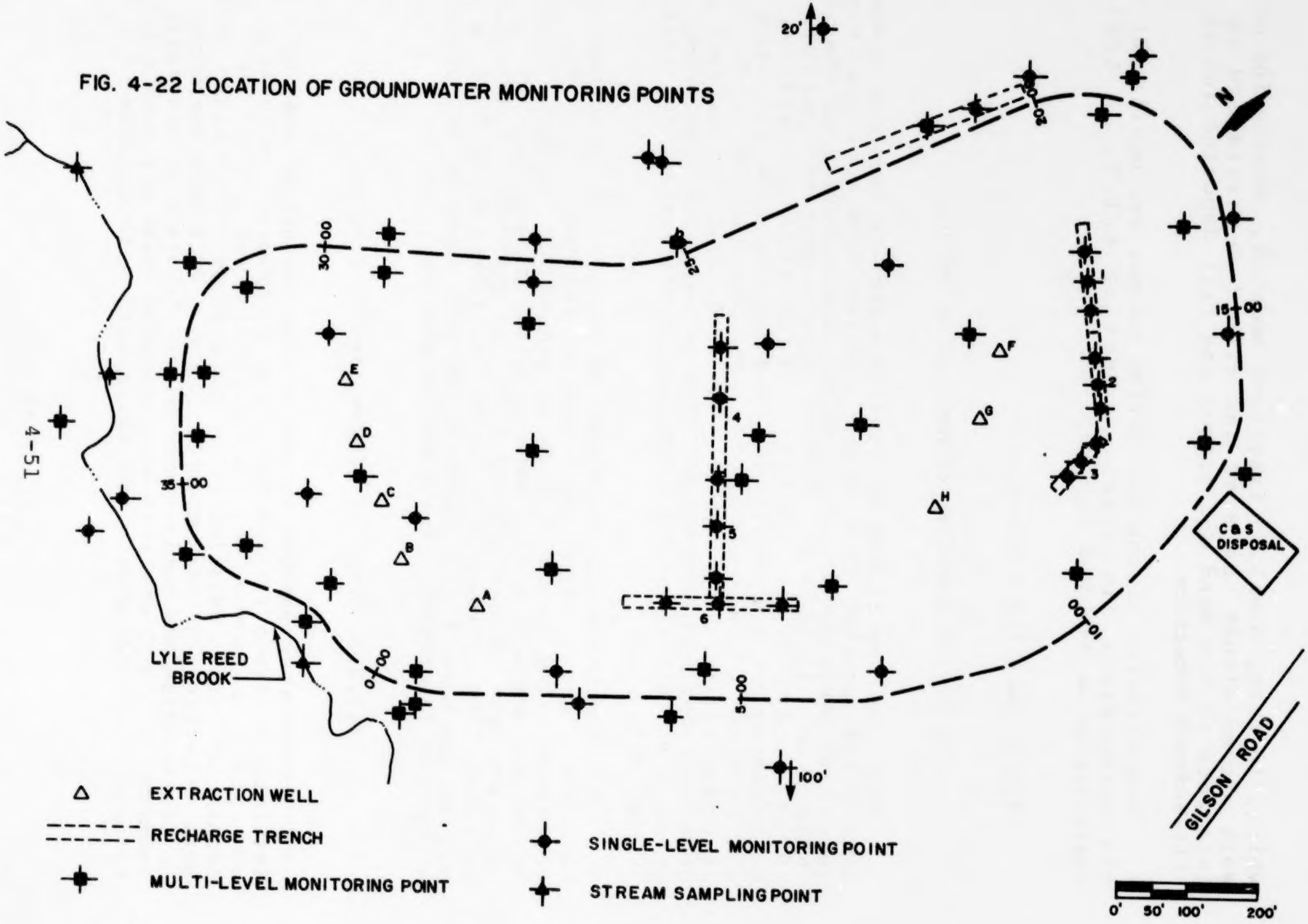
As part of the evaluation of the effectiveness of the cutoff containment wall installed at the Gilson Road site, groundwater samples were obtained and analyzed. The data generated by the groundwater sampling and analysis program provided direct evidence of remedial action performance by establishing contaminant location in temporal and spatial dimensions. The groundwater quality data, when used in concert with the hydraulic evaluations (Sections 4.1 and 4.2), aided in the evaluation of the long-term environmental impact and overall effectiveness of the cutoff wall containment.

Useful information, such as the determination of containment flow paths, was gained by comparing groundwater contamination data from monitoring points located within the same geologic strata but on opposite sides of the containment wall. Additionally, groundwater quality data collected over a relatively long period of time allowed evaluation of the effectiveness of milestone events which have occurred at the Gilson Road site.

4.3.1 Location and Installation of Groundwater Monitoring Wells

The installation of groundwater monitoring wells on the Gilson Road site dates back to 1979. Since that time both single and multi-level monitoring wells have been placed at strategic locations to provide data on the horizontal and vertical distribution of contaminants (see Figure 4-22). Monitoring wells installed by the state of New Hampshire in 1984 were placed primarily on the downgradient, or western, end of the site. The location of the monitoring points were selected so as to provide water quality data in similar geologic formations but on opposite sides of the cutoff wall. The geologic formations consist of a relatively thick sand and gravel deposit overlying a glacial till

FIG. 4-22 LOCATION OF GROUNDWATER MONITORING POINTS


N
C&S DISPOSAL
GILSON ROAD
0+00
5+00
10+00
15+00
20+00
25+00
30+00
35+00
A
B
C
D
E
F
G
H
1
2
3
4
5
6
20'
100'
LYLE REED BROOK
EXTRACTION WELL
RECHARGE TRENCH
MULTI-LEVEL MONITORING POINT
SINGLE-LEVEL MONITORING POINT
STREAM SAMPLING POINT
0'
50'
100'
200'

which, in turn, overlies fractured bedrock. Monitoring points were placed within the sand and gravel deposits and at the interface of the sand and gravel and till deposits and at the till-bedrock interface.

Installation of the monitoring points was consistent with the methodology discussed in Section 4.1.3. A typical installation is depicted in Figure 4-3.

4.3.2 Sampling and Analysis

4.3.2.1 Sampling of Monitoring Points

The sampling of the monitoring points to obtain groundwater samples has occurred since 1979. Samples were collected over a series of milestone events which have occurred on the site. These events include the initial site assessment period, the installation of an emergency recirculation system, the emplacement of a cutoff wall and impermeable cap, and the operation of a permanent groundwater hydrodynamic isolation/recirculation system. Over 330 chemical analyses were completed for the site during this period. Sampling rounds just prior to and subsequent to cutoff wall installation have included 70 analyses.

Observation wells were sampled using 1-inch stainless steel bailers. Three times the initial volume of the well was evacuated prior to collection of groundwater quality samples. Gas-driven sampling points were sampled using ultra-high purity nitrogen gas in accordance with the methodology outlined in Barvenik, 1984. Groundwater samples collected from the site were placed in ice-filled coolers and delivered to EPA certified analytical laboratories for chemical analyses.

4.3.2.2 Analysis of Groundwater Samples

Groundwater samples collected from The Gilson Road site were initially analyzed for the full range of priority pollutants. In addition, a number of non-priority pollutants were also identified. This data indicated the primary contaminants on-site were volatile organic compounds (VOCs). VOCs were therefore used to characterize and monitor contaminant levels both spacially and temporally. VOC analyses were executed using EPA Method 624 for priority list pollutants and equivalent methodologies for VOCs not found on the priority pollutant listing but known to exist on-site.

4.3.3 Results of Groundwater Quality Monitoring

4.3.3.1 Selection of Pertinent Groundwater Monitoring Points

As previously mentioned, a large number of groundwater monitoring points have been installed on the Gilson Road site. These points were sampled for groundwater quality information since 1979. As a result, a large volume of data exists regarding the distribution of contaminants on the site. In order to evaluate changes in groundwater quality in response to the installation of the cutoff wall, a subset of all of the groundwater monitoring points was chosen for detailed analysis. The monitoring points selected for this analysis satisfied the following criteria:

- The monitoring points were located at the downgradient portion of the site where head loss across the cutoff wall, and therefore resulting contaminant migration out of the site, is greatest;
- The monitoring points were composed of multi-level installations located within the three major geologic strata present on the site, whenever possible, in order to provide vertical contamination distribution information, and
- The monitoring points were sampled over an extended period of time covering the major milestone events which occurred during remediation of the site.

Based upon the above requirements, five groundwater monitoring points were chosen for detailed analysis. These five monitoring points show contaminant concentrations which are representative of the general trend in contaminant distribution of the downgradient portion of the site. In addition, a surface water sampling point located downgradient of the cutoff wall and along the monitoring point cross-section line was included in the analysis[7]. It should be noted that while the monitoring well concentrations are representative of the site in general, data gaps do exist throughout the sampling history. These data gaps were typically the result of changing conditions on the site during the remediation effort. Monitoring well placement and sample selection were primarily in response to specific data needs associated with remediation and as such were not always compatible with the cutoff wall evaluation effort.

The location of monitoring wells, as they ultimately relate to the cutoff wall evaluation, were influenced by several factors:

- As monitoring wells were destroyed and replaced over the relatively long history of the site, new locations for wells were selected so as to reflect crucial data requirements at the time of installation. As such, destroyed monitoring locations were not always replaced.

- Prior to the installation of the cutoff wall, there was no reason to place monitoring wells in close proximity to each other. Such an alignment only became productive once the wall was in place to allow comparison of contaminant levels inside and outside of the containment.

Given the large number of sampling points located on the Gilson Road site, not all monitoring points could be sampled during each round of data collection. The selection of sampling points was also influenced by several factors:

- The selection of sample locations was in response to the historic location of the contaminant plume on the site. The dynamics of the plume meant that different sampling points would be monitored so as to track plume movement;

- The selection of sample locations under the current contract was performed under a limited budget and was to meet multiple objectives. The groundwater quality data was collected for the purpose of cutoff wall evaluation and also to supply data to the State of New Hampshire for performance evaluation of the groundwater extraction and treatment system.

---

[7] In order to bridge the milestone events, data from two surface water sampling points were combined into one point. The two surface water monitoring points are located in close proximity to each other and show similar concentration levels and trends on dates where analyses were performed on both sample points.

The result of the competing factors for monitoring well placement and sampling location yields a history of groundwater quality on the site which contains some data gaps. However, the monitoring points chosen for detailed analysis give a general trend in contaminant distribution which can be used to evaluate the effectiveness of the cutoff wall. The monitoring point cross-section line is shown in Figure 4-23.

4.3.3.2 Selection of an Indicator VOC Compound

The selection of a single compound as an indicator for total VOCs allows for accurate comparison of data collected over an extended period of time. This allows the data to be less sensitive to changes in the priority and non-priority pollutant VOC list for which the samples were analyzed. Since a large percentage of the total VOCs detected on the site were non-priority pollutant VOCs, the groundwater samples were not always analyzed for the same suite of compounds. Therefore, the comparison of data collected over a long period of time is more accurate in this case through the use of an indicator compound than through comparison of total VOC concentrations.

The results of the chemical analyses for VOCs on groundwater samples from the Gilson Road site indicated the presence of one VOC at very high levels in almost all samples. Tetrahydrofuran (THF) comprised more than 90 percent of the VOCs present in most of the groundwater samples collected from the site. This compound was selected an an indicator for total VOC distribution on the site for the following reasons:

- Tetrahydrofuran, in most samples, comprises a large percentage of the total VOCs present in the sample. In most cases it is present in excess of 90 percent of the total VOCs;
- THF is relatively soluble in water and is not readily absorbed by soil particles; and
- THF is not significantly biodegradable in the subsurface environment.

As a result, the compound makes a good indicator for contaminant distribution and allows for comparison of groundwater quality data over relatively long periods of time. This data is summarized in Table 4-6.

FIG. 4-23 LOCATION OF SELECTED GROUNDWATER MONITORING POINTS

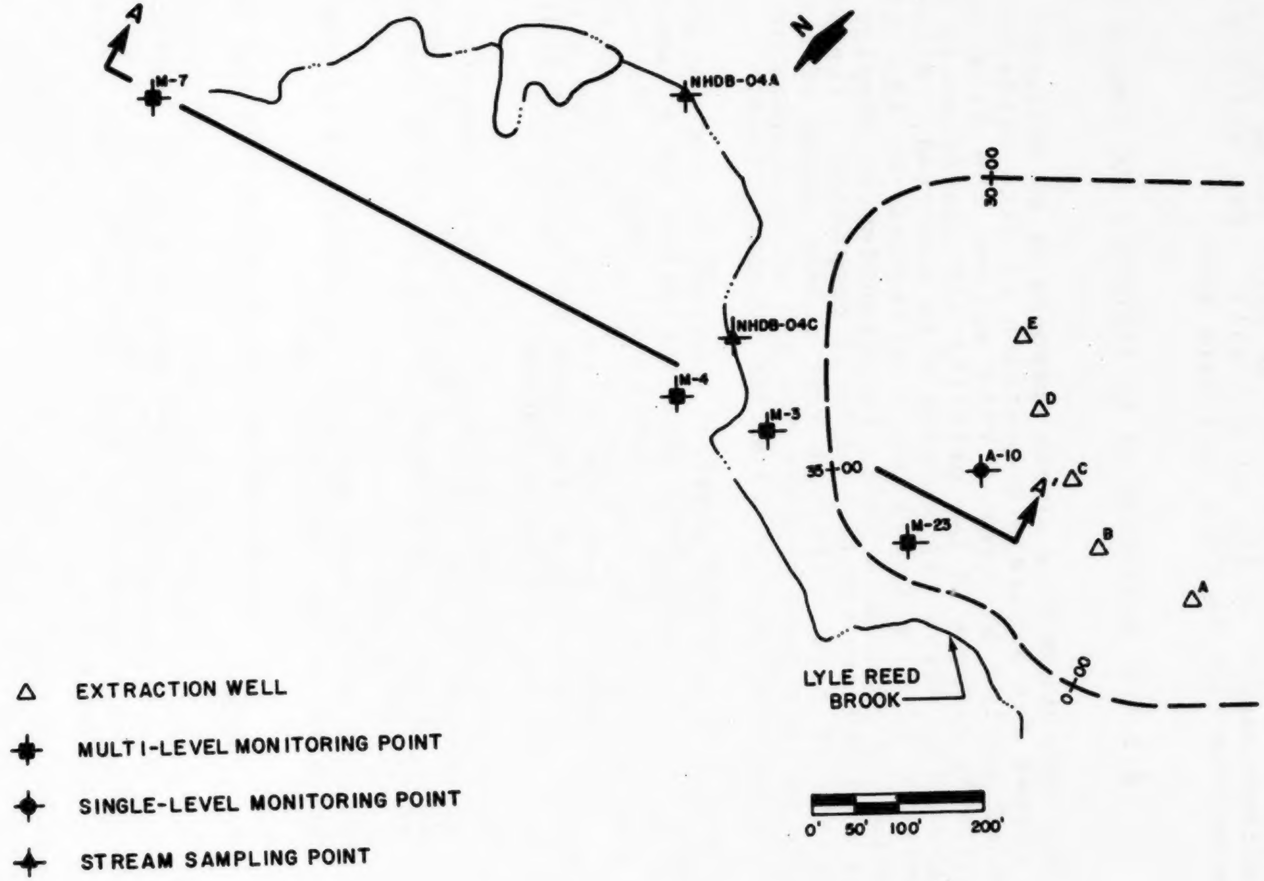

A
N
M-7
NHDB-04A
30+00
NHDB-04C
M-4
M-3
E
D
A-10
35+00
A'
C
M-23
B
A
LYLE REED BROOK
0+00
0' 50' 100' 200'
EXTRACTION WELL
MULTI-LEVEL MONITORING POINT
SINGLE-LEVEL MONITORING POINT
STREAM SAMPLING POINT

TABLE 4-6

CONCENTRATIONS OF TETRAHYDROFURAN IN PARTS PER MILLION

Well Location	Sampling Date 7/81	1/82	3/82	5/82	7/82	11/82	4/83	1/85*	10/85
A-10	--	850	572	46	178	55	--	--	--
M-23-1	--	--	--	--	--	--	59	--	0.045
M-23-2	--	--	--	--	--	--	150	--	10.0
M-23-4	--	--	--	--	--	--	83	--	80
M-3-1	--	16	3.2	5.0	0.89	--	--	--	ND
M-3-2	--	26	25.1	17.8	1.8	--	84	0.04	--
M-3-3	--	68	34.2	--	2.2	12	17	0.69	--
M-3-4	--	170	--	--	--	68	1.4	5.0	--
NHDB-04	12	--	1.2	1.3	0.4	0.9	1.9	0.3	--
M-4-1	--	54	--	--	--	3.7	--	--	--
M-4-2	--	34	--	--	--	33	110	--	--
M-4-3	--	4.4	--	--	--	1.3	--	ND	0.06
M-4-4	--	--	--	--	--	46	110	35	80
M-7-2	--	--	10	--	--	8.8	76	--	85
M-7-4	--	--	5.2	--	--	--	10	--	80

-- Indicates data not taken
*QA/QC data indicates a possible laboratory dilution error.

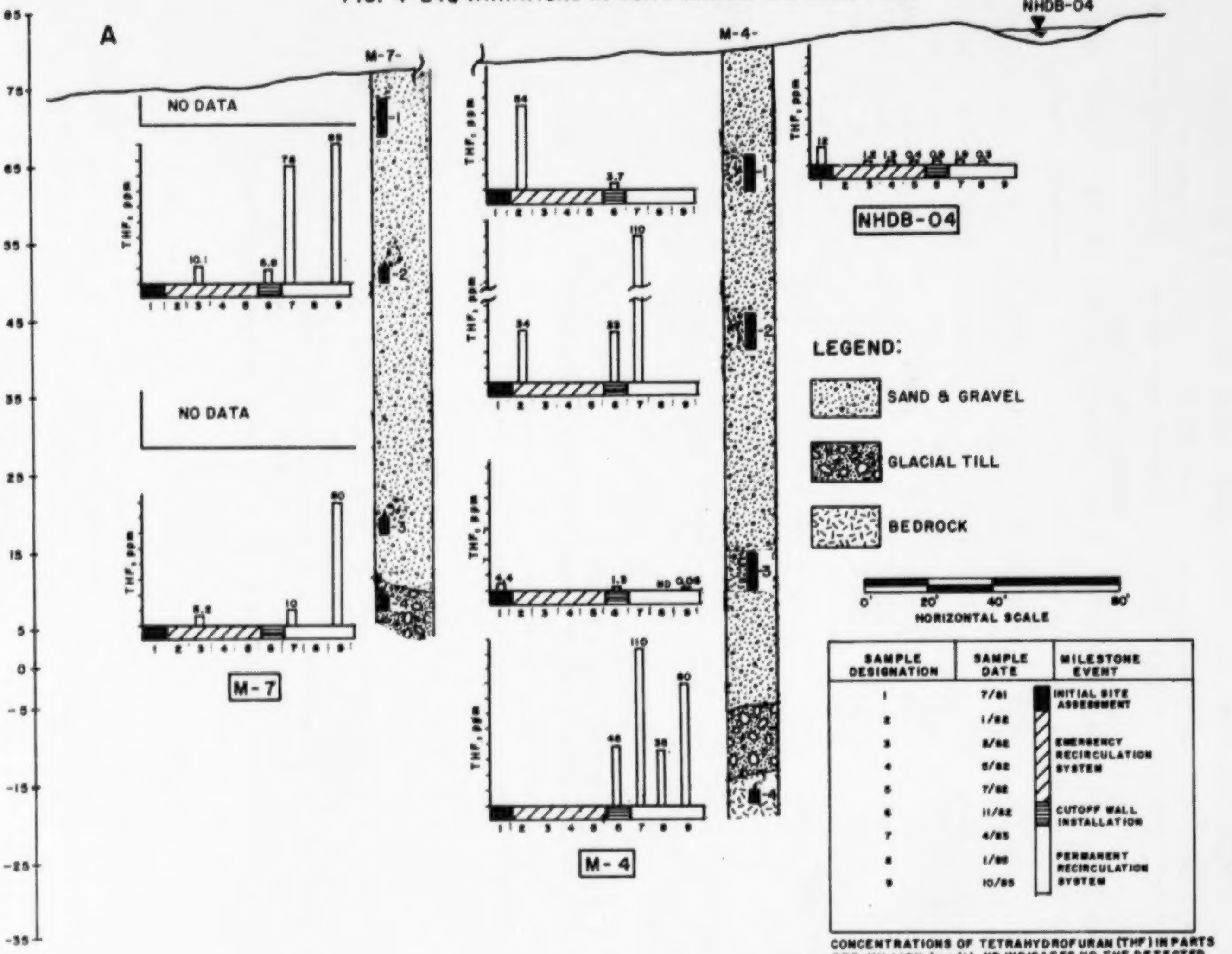
4.3.3.3 Data Discussion

The distribution of THF for the selected groundwater monitoring points is displayed both spatially and temporally in Figures 4-24 a and b. An examination of the data contained in the figures reveals several trends in the distribution of the contaminants in response to the milestone events which occurred on the site.

A. Contaminant Distribution During Initial Site Assessment

Contaminant distribution on the site, as indicated by the initial site assessment data, shows the movement of contaminants from the center of the site towards Lyle

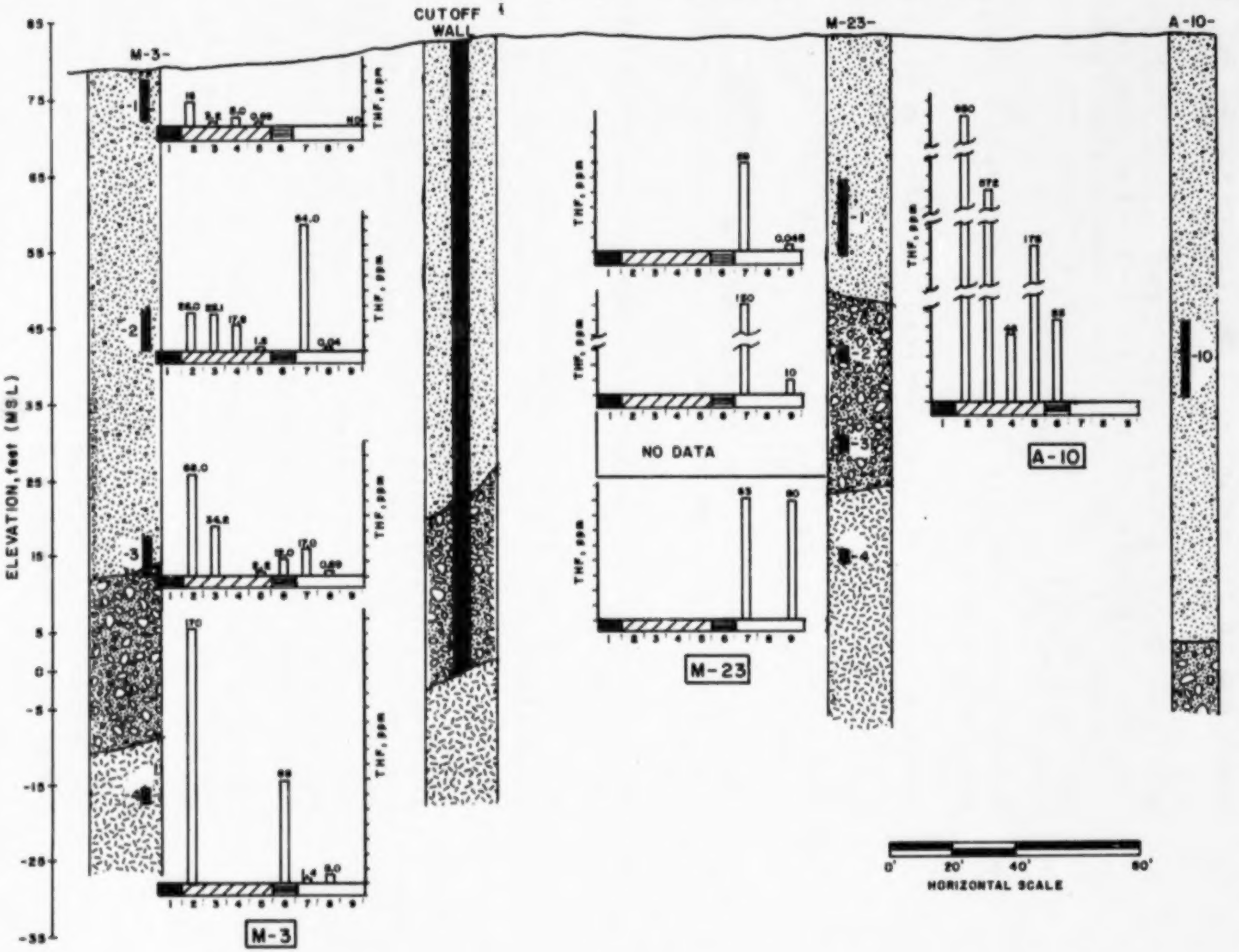
FIG. 4-24a VARIATIONS IN CONTAMINANT DISTRIBUTION


A
ELEVATION, feet (MSL)
85
75
65
55
45
35
25
15
5
0
-5
-15
-25
-35
M-7-
M-4-
NHDB-04
NO DATA
NO DATA
THF, ppm
10.1
6.8
78
88
6.2
10
90
M-7
84
3.7
34
53
110
4.4
1.3
ND
0.06
46
110
38
80
M-4
12
1.2
1.3
0.4
0.9
1.9
0.3
NHDB-04
1
2
3
4
5
6
7
8
9
-1
-2
-3
-4
LEGEND:
SAND & GRAVEL
GLACIAL TILL
BEDROCK
0'
20'
40'
80'
HORIZONTAL SCALE

SAMPLE DESIGNATION	SAMPLE DATE	MILESTONE EVENT
1	7/81	■ INITIAL SITE ASSESSMENT
2	1/82	▨ EMERGENCY RECIRCULATION SYSTEM
3	3/82	▨
4	5/82	▨
5	7/82	▨
6	11/82	☰ CUTOFF WALL INSTALLATION
7	4/85	□ PERMANENT RECIRCULATION SYSTEM
8	1/86	□
9	10/85	□

CONCENTRATIONS OF TETRAHYDROFURAN (THF) IN PARTS PER MILLION (mg/l). ND INDICATES NO THF DETECTED.

FIG. 4-24b VARIATIONS IN CONTAMINANT DISTRIBUTION


A'
CUTOFF WALL
M-3-
M-23-
A-10-
ELEVATION, feet (MSL)
85
75
65
55
45
35
25
15
5
0
-5
-15
-25
-35
-1
-2
-3
-4
-10
THF, ppm
1 2 3 4 5 6 7 8 9
16
5.2
8.0
0.89
N.D.
26.0
22.1
17.8
1.5
64.0
0.04
62.0
34.8
5.6
12.0
17.0
0.89
170
68
1.4
9.0
M-3
68
0.048
120
10
NO DATA
53
90
M-23
980
672
32
178
52
A-10
0'
20'
40'
80'
HORIZONTAL SCALE

Reed Brook, on the northwest boundary of the site. This movement is reflected in increasing concentrations of contaminants in water samples collected both from the brook and in the vicinity of the future cutoff wall.

Concentrations in Lyle Reed Brook peaked at 12 parts per million during the July 1981 sampling round. Concentrations of THF at M-3 also showed peak concentrations during this time frame ranging from 16 to 170 parts per million at the various sample levels (concentrations at M-3 increased with depth).

B. Contaminant Distribution in Response to Emergency Recirculation System

Concentrations at the downgradient portion of the site began to decrease after the start-up of the Emergency Recirculation System. Levels decreased during operation of the system (February, 1982 through November, 1982) in the stream sampling point, NHDB-04, and in the downgradient wells, M-3 and A-10. This response in concentrations to recirculation system operation shows the system retarded the advance of the contaminant plume by recirculating water back to the central portion of the site.

C. Contaminant Distribution in Response to Installation of Cutoff Wall

The response of contaminant distribution to the installation of the cutoff wall were twofold. Just prior to installation of the cutoff wall at the downgradient portion of the site, the emergency recirculation system was shut down in order to allow grading, pumping well extension, and impermeable cap placement. As a result, the advancement of the plume, which had been halted and reversed during the functioning of the system, once again began. This plume movement was reflected in contaminant levels at the downgradient monitoring wells.

The cutoff wall was completed in November, 1982. Monitoring points located downgradient of the wall continued to show an increase in concentration. This increase was in response to movement of contaminants beyond the cutoff wall location while the emergency recirculation system was shut down. This pulse of contaminants continued to be observed over time as it moved downgradient. In general, at the time of the completion of the cutoff wall, the levels of contaminants were highest within the bedrock.

With the completion of the wall, contaminant levels in the area of the stream again decreased. In addition, concentrations inside the cutoff wall remained higher than levels outside the cutoff wall. Contaminant levels in the vicinity of the cutoff wall were highest in the bedrock. These levels indicate flow in the vicinity of the cutoff wall was occurring within the fractured bedrock and not through the wall itself.

D. Contaminant Distribution in Response to the Permanent Recirculation System

Currently, contaminant levels within the cutoff wall have decreased as compared to levels measured subsequent to cutoff wall completion. This decrease is primarily due to the functioning of the permanent recirculation system. Pumping of groundwater within the cutoff wall has had the effect of mixing or homogenizing the groundwater. As a result, levels within the cutoff wall have dropped as highly contaminated zones were mixed with zones exhibiting lower concentrations. However, even with the homogenizing effect, levels within the cutoff wall are higher than those outside of the wall.

4.3.3.4 Contaminant Degradation of the Cutoff Wall

-The cutoff wall installed at Gilson Road has performed as a multi-functional containment structure. Initially, the wall served as a temporary measure to impede the off-site migration of contaminants. With the start up of the permanent hydrodynamic isolation system, the cutoff wall now functions as a clean water exclusion barrier to inhibit the flux of clean water back onto the site in response to groundwater pumping. While performing the initial containment function, the wall was exposed to contaminants. As a result, the potential for chemical degradation of the wall was investigated.

Two major factors control the time required for cutoff wall degradation:

- The number of pore volume displacements required to affect a chemically mediated change in the wall hydraulic conductivity;
- The rate at which leachate flows through the wall.

Long-term hydraulic conductivity testing of the Gilson Road backfill with worst case leachate from the site indicated that displacement of two to three pore volumes are required to effect changes in the cutoff wall hydraulic conductivity (Schulze, 1984). The testing, which simulated in-situ conditions, indicated a maximum increase to twice the initial hydraulic conductivity (1.2×10^{-8} to 2.5×10^{-8} cm/sec). These results agree with literature documentation on chemical degradation of clay based barriers.

The flow of contaminated groundwater through the cutoff wall intact backfill is dependent upon the hydraulic gradient (i) across the wall and the intact hydraulic conductivity of the wall (k). The hydraulic gradient is determined by the ratio of the total head difference across the wall as compared to the width of the wall (L). The intact hydraulic conductivity of the wall was determined by hydraulic conductivity testing of cutoff wall samples (Section 2).

Flow through a porous medium is governed by Darcy's Law ($Q = kiA$) where Q is the volume of water flowing through a cross-sectional area of the medium (A). The average velocity of the flow through the medium is described by the equation $V_s = ki/n$, where n is the porosity of the material. The time required to displace one pore volume can be determined by the equation $t = W/Vs$, where W is the width of the medium through which flow is occurring.

In order to determine the time to displace one pore volume of water through the wall, certain conservative, or worst-case, assumptions were made in the selection of parameter values.

The greater the hydraulic gradient (i) across the wall, the faster the wall will degrade. The maximum head difference observed on the site was three feet. This condition only occurred during one period of particularly high precipitation without hydrodynamic controls. A minimum width of the wall would be equal to the width of the bucket used to excavate the cutoff wall trench. A 3 foot bucket was used. This results in a maximum hydraulic gradient of $i = 1$.

The greater the hydraulic conductivity of the wall, the faster degradation should occur. The hydraulic conductivity of the wall is inversely proportional to the level of stress on the wall. The stress on the wall increases with depth to a maximum of 50 pounds per square inch (psi) at the bottom of the wall (105 feet). In order to simulate worst case conditions, hydraulic conductivity testing of the cutoff wall was conducted using stress conditions corresponding to the top of the cutoff wall, or

3 to 5 psi. Although design phase laboratory testing and construction QC testing in the field yielded an average cutoff wall hydraulic conductivity of 5×10^{-8} cm/s, a more conservative value of 1×10^{-7} cm/s was used for determining pore volume displacement rates as based on work completed under phase one (Section 2) of this contract.

The porosity (n) of a material is that portion of the material not occupied by solid matter relative to its total volume. If all other factors remain constant, a lower porosity will result in a higher flow velocity, which will cause a faster degradation of the cutoff wall. Based on QC data, a conservative value of $n = 0.4$ was chosen.

A computation of the time to displace a single pore volume is determined using the equations stated previously:

$$t = \frac{W}{Vs}; \text{ where}$$

$$Vs = \frac{ki}{n}$$

Using the conservative, or worst case, values stated above, the form of the equations becomes:

$$Vs = \frac{1 \times 10^{-7} \text{ cm/s } (1)}{0.4} = 2.5 \times 10^{-7} \text{ cm/s.}$$

$$t = \frac{91.44 \text{ cm}}{2.5 \times 10^{-7} \text{ cm/s}} = 3.7 \times 10^{8} \text{ seconds;}$$

$$t = 11.6 \text{ years}$$

The time to displace one volume of pore water using worst case parameters is calculated to be approximately eleven years. However, under the average conditions which have actually occurred on the site, the time required to displace a single pore volume is approximately fifty years. As the testing of the degradation of the cutoff wall material has shown, chemical degradation of the material was complete only after two pore volume displacements, thereby doubling the likely time estimates quoted above.

It is realized that chemical degradation of the wall would begin immediately at the surface in contact with the leachate. As a result, the hydraulic conductivity of the backfill at the surface, over a thickness dx, would increase. Assuming as a worst case that the increase was infinite, then the gradient across the wall would increase commensurate with the decrease in effective wall width. Integration of the appropriate equations demonstrates that the rate of chemical degradation would increase by a factor of two as compared to that computed assuming constant gradient. The assumption of infinite increase in hydraulic conductivity due to chemical degradation, however, is conservative in that actual testing indicates only a two fold increase. As such, the increase in rate of degradation due to a changing gradient would be less than two.

Given the data and computations performed above, it is unlikely that the Gilson Road cutoff wall has undergone significant chemical degradation in the 2.5 years since its construction began. The calculation of potential cutoff wall degradation assumed a hydraulic gradient of 1. However, the operation of the hydrodynamic isolation system, in place since April 1985, has acted to balance the hydraulic heads inside and outside the wall, thereby reducing the hydraulic gradient to essentially zero. In addition, the operation of the groundwater treatment plant under construction at the site includes a treatment purge stream located outside the cutoff wall. As a result, pumping of groundwater within the wall will cause a water deficit within the cutoff wall and result in flux into the site. This flux into the site will cause relatively clean groundwater to flow through the wall, thereby reversing the degradation caused by the flow of leachate through the wall if any has actually occurred.

4.4 Conclusions

The hydraulic stress testing and contaminant migration analyses both indicate that the cutoff wall appears to be functioning as an essentially intact barrier. However, it is apparent that the fractured bedrock which forms the bottom of the containment is highly pervious and would result in a major leakage path without the hydrodynamic isolation systems incorporated in the overall containment design. The following more specific conclusions can be drawn from the Phase Three work as summarized herein.

- The three-dimensional numerical modeling of the site predicted that the bedrock located at the downgradient portion of the containment was more pervious than initially indicated via packer testing data. The bedrock pumping test verified this prediction, yielding bedrock hydraulic conductivities in the range of 10^{-1} cm/sec. This value is large as compared to that specified for the cutoff wall (1×10^{-7} cm/sec). The pumping test also demonstrated that the glacial till existing just above the bedrock was also quite pervious and probably discontinuous. This data supports conclusions reached during the RI/FS.

- The cutoff wall and bottom aquitard/aquifer units form a hydraulically coupled system. Analysis of cutoff wall bulk hydraulic conductivity therefore must rely on numerical modeling to correlate the stress test data and separate the behavior of the wall from that of the bedrock. The accuracy of the bulk hydraulic conductivity computed via the sensitivity analysis is therefore inherently limited to the calibration accuracy of the numerical model. As such, post-construction verification efforts are based on somewhat circumstantial data in the form of piezometric head distributions and thus could be opened to varying interpretations.

- Analyses of cutoff wall bulk hydraulic conductivity must be based on hydraulic stress testing of the containment. The value obtained will therefore inevitably be in the form of an upper bound solution. The proximity of the upper bound value obtained via analysis to that specified for the cutoff wall will be limited by not only the success of the cutoff wall construction effort, but also by the hydraulic conductivity of the containment bottom. The very cases which are likely to require post-construction verification studies (containment leakage) are therefore those for which hydraulic stress analysis may be the least conclusive. In these instances, specifications based solely on performance criteria may prove difficult to enforce.

- The bulk hydraulic conductivity of the Gilson Road cutoff wall was found to be less than 10^{-5} cm/sec. The degree to which the actual value falls below this upper bound cannot be determined due to the high hydraulic conductivity of the containment bottom. However, a worst case value of 10^{-5} cm/sec yields a cutoff wall efficiency of greater than 93%. The actual value of wall hydraulic conductivity is probably approximately 1×10^{-7} cm/sec as based on quality control testing (Section 2). This value yields a cutoff wall efficiency in excess of 99%.

- The overall passive containment efficiency, including bedrock leakage, is also important. The overall efficiency of the passive containment elements (cutoff wall, cap and fractured bedrock) was found to be approximately 55% (under worst case precipitation induced stress conditions) with the major loss flowing out through the bedrock aquifer. It was recognized during the RI/FS phases of the project that the glacial till/bedrock aquitard/aquifer containment bottom would leak. However, additional passive barriers such as grouting to stop such leakage were not found to be economically feasible. Hydrodynamic isolation systems were therefore incorporated in the overall containment design. Hydrodynamic isolation not only proved more cost efficient than additional physical barriers, such as grouting the bedrock, but also provided for recirculation/treatment capabilities. Following this approach, a purge stream pumpage was instituted in 1986 when the groundwater treatment system went on line. At this time, over 72,000 GPD are being discharged outside the cutoff wall in order to implement the hydrodynamic system elements.

° The contaminant migration analysis supported the overall conclusions derived form the hydraulic stress testing. The data indicated that contaminant flux, in the absence of hydrodynamic isolation, would be through the bedrock below the wall. However, the contaminant data was inconclusive when analyzed independently. This stems from the lack of sufficient data due to cost and conflicting objectives governing sample selection as well as delays in the start-up of the hydrodynamic isolation system due to construction contract difficulties.

° The data obtained and the computations executed during this study in combination with long-term leachate/backfill compatibility testing undertaken as part of the cutoff wall design process indicate that significant chemical degradation of the cutoff wall is unlikely over the 2.5 years since its construction. The two pore volume exchanges required for chemical degradation are computed to take over 20 years under worst case conditions. Leachate/backfill compatibility testing indicates that the worst case leachate found at the Gilson Road site only increases the hydraulic conductivity of the intact backfill by a factor of two.

SECTION 5

CONCLUSIONS

The following general conclusions can be drawn from the work executed under this contract. More specific and detailed conclusions with respect to the individual work phases can be found at the end of their respective sections.

- Work on the Gilson Road project demonstrated that a high degree of emphasis on field quality control procedures was required to achieve and document satisfactory cutoff wall installation. To be meaningful, the groundwork for the field QC testing should be initiated during backfill mix design and leachate compatibility testing. The results of this design phase work should be reflected in the project construction specifications as minimum design standards. QC testing should then be executed in the field during construction with emphasis on immediate data availability such that it can be used to guide the construction on a day to day basis.

- The API fixed-ring hydraulic conductivity and methylene blue titration procedures, developed for quality control of the Gilson Road cutoff wall, proved to be highly applicable and practical testing methods. The hydraulic conductivity and percent bentonite content data that these tests provided were sufficiently precise and accurate for evaluation of conformance to the project specifications. As such, these tests should be routinely incorporated into the quality control program for soil/bentonite cutoff walls used to contain hazardous wastes.

- Modified piezocone instruments appear to be capable of detecting windows in soil/bentonite backfilled cutoff walls. However, the present state of the art requires significant judgment with respect to data interpretation. This judgment must be based on a thorough understanding of the equipment used and general soil mechanics theory. Prior to routine use as a post-construction verification tool, a significant amount of additional work must be completed to build up a data base upon which empirical correlations can be constructed.

- For piezocone sounding or direct sampling to be viable as post-construction verification tools, construction quality control of the cutoff wall must include wall alignment and verticality provisions. The accuracy of as-built wall alignment documentation and actual wall verticality must be increased significantly above that required for hydraulic performance only.

- Post-construction verification of cutoff wall bulk hydraulic conductivity is an inherently costly endeavor. Under most actual field conditions of interest, sensitivity analyses, based on calibrated 3-dimensional numerical models, will be required to evaluate hydraulic stress testing data. The value of bulk hydraulic conductivity obtained for the cutoff wall will therefore typically be derived from somewhat circumstantial evidence and will be inconclusive in cases where the bottom aquiclude is pervious as compared to the value specified for the cutoff wall. Therefore, use of quality control testing during construction is typically preferable to reliance on post-construction verification for enforcement of project specifications.

- Based on the work performed under this contract, the Gilson Road cutoff wall appears to behave as an essentially intact barrier to flow. As such, it is performing as intended with a lower bound efficiency of greater than 93 percent (an actual efficiency of over 99 percent is probable as based on hydraulic conductivity values obtained from quality control testing). However, the fractured bedrock, which forms the bottom of the containment, appears to be more pervious than initially anticipated. As such, pumping rates for the hydrodynamic isolation/recirculation system, incorporated as part of the overall containment system to account for the fractured rock, will likely have to be increased relative to initially predicted values.

- Work on the Gilson Road project has demonstrated that cutoff walls can be successfully constructed for hazardous waste containment. These physical barriers can prove highly cost efficient as an integral component of the overall permanent containment system. This is particularly true for sites where the objective is to eliminate, rather than just reduce, off-site contaminant migration. In these cases, the cutoff wall should be viewed as a clean water exclusion barrier used to increase hydrodynamic isolation/recirculation effectiveness and reduce treatment costs.

REFERENCES

Alther, George. Design, Testing & Construction of Bentonite Liners. International Minerals & Chemical Corporation, (undated).

API. American Petroleum Institute. API Recommended Practice: Standard Procedure for Testing Drilling Fluids. API RP13B. American Petroleum Institute, Dallas, Texas, 1982.

API. American Petroleum Institute. API Specification for Oil Well Drilling Fluid Materials. API Spec. 13A American Petroleum Institute, Dallas, Texas, 1981.

Ayres, J. E., and M. J. Barvenik, and D. C. Lager. The First EPA Superfund Cutoff Wall; Design and Specifications. Proceedings of the 3rd National Symposium and Exposition on Aquifer Restoration and Groundwater Monitoring, sponsored by NWWA, Columbus, OH, May 1983.

Baligh, M. M., and J. N. Levadoux. Pore Pressure Dissipation After Cone Penetration. Publication No. R80-11, Order No. 662, Department of Civil Engineering, MIT, Cambridge, MA, April 1980.

Baroid/HL Industries. Field Testing of Drilling Muds, NL Baroid/NL Industries, Inc., Houston, Texas (undated).

Barvenik, M. J. Dedicated Gas-Drive Groundwater Sampling Instruments, Lecture and Notes presented as part of the Design, Installation, and Sampling of Groundwater Monitoring Wells Short Course sponsored by the National Water Well Association, multiple courses in 1984.

Barvenik, M. J. Methods for Passive Physical Containment of Groundwater Contamination and Field Applications of Active and Passive Physical Containment of Contaminated Groundwater. Lectures and notes presented as part of the Short Course entitled: Corrective Action for Containing and Controlling Groundwater Contamination, sponsored by the National Water Well Association; multiple courses in 1985 and 1986.

Barvenik, M. J. and R. M. Cadwgan. Multi-level Gas-Drive Sampling of Deep Fractured Rock Aquifers in Virginia, Groundwater Monitoring Review, Vol. 3, No. 4, Fall 1983.

Brown, D. M., and L. W. Gelhar. The Hydrology of Fractured Rocks: A Literature Review. MIT Parsons Laboratory Technical Report #304, 1986.

Cedergren, H. R. Seepage, Drainage, and Flow Nets. John Wiley & Sons, Inc., NY, NY, pp. 6, 1967.

Douglas, B. J. and R. S. Olsen. Soil Classification Using Electric Cone Penetrometer. Sym. on Cone Penetration Testing and Experience. Geot. Eng. Div., ASCE, St. Louis, Missouri, Oct. 1981.

Hantush, M. S. Flow to Wells in Aquifers Separated by a Semi-pervious Layer, Journal of Geophysical Research, 72 (6), 1709-1720, 1967.

JRB. Slurry Trench Construction for Pollution Migration Control. U.S. EPA Document EPA-540/2-84-001, Cincinnati, OH, 1984.

Jones, G. A., and D. Van Zyle, and E. Rust. Mine Tailings Characterization by Piezometer Cone. Sym. on Cone Penetration Testing and Experience. Geot. Eng. Div., ASCE, Oct. 1981.

Koteff, C. and R. P. Volchman. Surficial Geologic Map of the Pepperell Quadrangle. USGS, GP-1118, 1973.

Lambe, T. W., and R. V. Whitman. Soil Mechanics. John Wiley & Sons, Inc., NY, NY, 1969.

Levadous, J. N. and M. M. Baligh. Pore Pressures During Cone Penetration in Clays, Research Report. Department of Civil Engineering, MIT, Cambridge, MA, 1980 (in preparation).

Middlebrooks, T. A. Earth Dam Practice in the United States. Trans. ASCE (centennial volume), pp. 697, 1953.

Poulos, H. G. and E. H. Davis. Pile Foundation Analysis and Design. John Wiley & Sons, Inc., NY, NY, 1980.

Powers, M. A., and P. P. Virgadamo, and W. E. Kelly. Groundwater Control at Hazardous Waste Sites. ASCE National Convention on Hazardous Waste, St. Louis, Missouri, 1981.

Sanglerat, G. The Penetrometer and Soil Exploration. Elsevier Publishing Company, Amsterdam-London-New York, 1972.

Schmertmann, J. H. Guidelines for CPT Performance and Design. U.S. Dept. of Transportation, FHWA-T5-78-209, Feb. 1977.

Schulze, D., and M. J. Barvenik, and J. E. Ayres. Design of the Soil/Bentonite Backfill Mix for the First EPA Superfund Cutoff Wall. Presented at the Fourth National Symposium on Aquifer Restoration and Groundwater Monitoring, sponsored by NWWA, Columbus, OH, 1984.

Torak, L. J. Modifications and Corrections to the Finite-Difference Model for Simulation of Three-Dimensional Groundwater Flow. USGS Water Resources Investigation, 82-4025, 1982.

Trescott, P. C. Documentation of Finite-Difference Model for Simulation of Three-Dimensional Groundwater Flow. USGS Open File Report 75-438, 1975.

U.S. Army. Laboratory Soils Testing. Department of the Army, Manual EM 1110-2-1906, November 1970.

Winterkorn, H. F., and H. Y. Fang. Foundation Engineering Handbook. Van Nostrand Reinhold Company, NY, NY, 1975.

Wissa, A. E. Z., and R. T. Martin, and J. E. Garlanger. The Piezo Probe, Proceedings. ASCE Specialty Conference on In-Situ Measurement of Soil Properties, Raleigh, NC, Vol. I, pp. 536-545, 1975.

APPENDIX A

API FIXED-RING PROCEDURE

Purpose

The purpose of this procedure is to determine the hydraulic conductivity of a sample of backfill using the standard API filter press apparatus.

Materials

1. Standard filter press and mud cell assembly (as manufactured by Baroid, refer to Figure 2-1 of text)
2. Pressure source and feed lines
3. Pressure gage
4. Pressure regulator
5. Graduated cylinder (10 millileter capacity marked at 0.1 ml intervals)
6. Dry bentonite
7. Saturated Ottawa sand
8. Paste trimmer
9. Spatula
10. Analytical scale (accuracy: 0.01 grams)
11. Drying oven
12. Approximately 600 grams of backfill sample

Test Procedure

1. With a spatula, smear a paste consisting of bentonite and water on the inside of the API cell. The paste should be dry enough to adhere to the sides of the cell, but wet enough to saturate the dry bentonite. The purpose of the paste is to create a seal between the edge of the backfill sample and the inner surface of the cell. Be careful to fully cover the wall over the 2-inch high area to be occupied by the backfill.

2. Place the cell in the paste trimmer. Adjust the trimming edge so a bentonite seal of approximately 1/32-inch will be obtained. Evenly rotate the cell in the trimmer to remove the excess bentonite paste from the cell walls. Remove the cell from the trimmer and check if a 2-inch high zone starting 1/4-inch up from the bottom of the cell is entirely covered by a smooth, 1/32-inch bentonite seal. The paste will line the inside of the cell over the section occupied by the backfill sample.

3. With a spatula and paper towel, remove any excess bentonite on the cell wall, outside of the area to be in contact with the backfill.

4. Assemble the base cap with filtrate tube, rubber gaskets, steel mesh screen, and cell.

5. Carefully place saturated Ottawa sand in the cell to a depth of 1/4-inch. The sand should just be in contact with the bottom edge of the bentonite seal. This sand will act as a lower filter. Sand is most evenly spread by blocking the drainage channel with a finger, submerging the sand (in water), and tapping the outside of the cell to smooth the surface. Outlet is then opened to let excess water drain off. If backfill mix is such that migration of fines and possible subsequent "blow-through" is a concern, carefully place a 2-inch diameter filter paper on top of sand. (Fines on the filter paper are also a good indicator of migration after test is completed).

6. Prepare the backfill sample for testing. Pass the wet backfill sample through the 1/2-inch sieve. On an analytical scale, weigh the test sample which should consist only of material passing the 1/2-inch sieve. Using the spatula, place the sample material into the cell. The sample should be placed so as to leave no air voids and without scraping the bentonite paste seal. After 2 inches of material has been placed, smooth the sample surface using the spatula.

7. Weigh the portion of backfill test material not placed in the cell and record the number of grams of backfill placed in the cell. With the remaining material, determine the moisture content of the sample at preparation.

8. Place the cell with the prepared soil sample into the filter press frame. Slowly, fill the cell above the soil sample with water. Do not disturb the fine materials at the sample surface while adding the water. This can be facilitated by placing a piece of filter paper on top of the backfill mix, then slowly adding water with a lab squeeze bottle on top of the filter paper. This will reduce the chance of sample degradation by scouring. Be sure to remove filter paper when cell is full (it should float to the top). The water in the cell should remain clear.

9. Place the top cap and its rubber gasket on the cell. Clamp the unit in place with the T-screw.

10. Set an empty graduated cylinder under the filtrate tube.

11. Apply a "seating" pressure to the sample corresponding to the lowest setting of the regulator (typically 1-1.5 psi).

12. Increase the pressure to 2 psi. Let sit for thirty minutes, then increase to 3 psi.

13. Read the water level in the graduated cylinder every two hours for six to eight hours. Record the time, the water level, the time increment since the last reading (in minutes), and the change in water level since the last reading (in cubic centimeters). Calculate the permeability for each time increment (using change in water volume and elapsed time since last reading). Also, calculate flow and volume of water in head space to determine when sample must be refilled (i.e., particularly important for overnight testing).

14. If the permeability has stabilized after the initial two hours water level readings, take readings every eight to fourteen hours. If the permeability is fluctuating, continue to take water level readings frequently (every two hours) until stabilization.

15. Run the test for a minimum of twenty-four hours.

16. If test data at a higher pressure desired, increase the cell pressure and repeat steps 13-15.

17. Keep a record of the total volume of water permeating the soil sample. Refill the cell above the soil sample with additional water when needed.

APPENDIX B

METHYLENE BLUE TEST PROCEDURE

I. PURPOSE OF TEST

The methylene blue test is a technique used to measure the cation exchange capacity of a soil/bentonite mixture. The purpose of the test is to measure bentonite content in soil/bentonite backfill utilizing the proportionality which exists between bentonite content and cation exchange capacity.

II. GENERAL PROCEDURE

A sample of soil and bentonite weighing 10 grams is mixed with a solution of the dispersant tetrasodium pyrophosphate (TSPP). The TSPP disperses the bentonite and replaces exchangeable cations of bentonite with sodium which facilitates the exchange process. A solution of methylene blue in distilled water is added to the slurry of soil/bentonite and dispersant in measured volumes until the exchange capacity of the sample is exceeded. At each increment in the titration, a drop of the slurry/methylene blue mixture is placed on a piece of filter paper. Prior to reaching the exchange capacity of the bentonite (endpoint), the drop appears as a uniformly dark blue circle. The end point is indicated by the appearance of a light blue halo around the dark blue dot. At this point, the exchange process is complete and free methylene blue cations are in solution (causing the halo).

III. PREPARATION OF METHYLENE BLUE SOLUTION

A. Materials and Apparati

1. Methylene blue crystals: U.S.P. grade, formula weight of 373.9, zinc free (available from EM Science)
2. Distilled water
3. Analytical balance (accuracy: 0.01 grams)
4. 1000 - milliliter volumetric flask
5. Funnel
6. Magnetic stirrer and stirring bar

7. Dark brown glass bottle with air-tight screw-on cap

B. Procedure

1. Weigh out 3.74 grams of methylene blue (MB) crystals,
2. Transfer the crystals to a 1000 - milliliter volumetric flask using a funnel,
3. Add approximately 500 milliliters of distilled water to the flask,
4. Stir the MB solution for one-half hour using the magnetic stirrer and stirring bar.
5. Remove the stirring bar from the solution and carefully fill the flask to the 1000 - milliliter calibration mark with distilled water,
6. Stir the MB solution for another half hour,
7. Transfer the solution to a dark brown glass storage container,
8. Repeat steps 1-7 for additional methylene blue solution (if desired),
9. Continuously stir the MB solution for twelve hours using the magnetic stirrer,
10. Label the storage container, including the date of preparation; store the MB solution at room temperature and out of direct light; the storage life is approximately three months.

IV. PREPARATION OF DISPERSANT SOLUTION OF TETRASODIUM PYROPHOSPHATE

A. Materials

1. Tetrasodium pyrophosphate (TSPP) crystals: specifically $Na_4 P_2 O_7 \cdot 10H_2O$
2. Distilled water
3. One liter flask
4. Analytical balance (accuracy: 0.01 grams)

B. Procedure

1. Weigh out 33.30 grams of the TSPP crystals,
2. Transfer the crystals to a flask containing one liter of distilled water,
3. Shake flask for five to ten minutes, until crystals are completely dissolved (determined visually).

V. PREPARATION OF SOIL SAMPLES

A. Materials

1. Soil sample and tare
2. Drying oven
3. Mortar and rubber pestle
4. No. 10 sieve
5. Analytical flask
6. 250 millimeter erlenmeyer flask
7. 2 percent solution of TSPP (prepared per Section IV)
8. Magnetic stirrer
9. Glass stirring rod
10. Hotplate

B. Procedure

1. Weigh a representative backfill sample,
2. Oven dry the sample,
3. Weigh the dry sample,
4. Using a mortar and rubber pestle, break up hardened clumps of dry soil,
5. Pass the sample through a No. 10 sieve,

6. Weigh the portion of the dry sample passing the No. 10 sieve.

7. Weight out 10.00 grams of the sample which passes the No. 10 sieve in a 250-ml erlenmeyer flask,

8. Add one hundred milliliters of 2 percent hydrated tetrasodium pyrophosphate (TSPP) solution to the 10.00 gram sample,

9. Mix the resultant slurry with a magnetic stirrer. Use a glass stirring rod to force any clay sticking to the flask walls into the solution,

10. On a hotplate, heat the sample to a slow rolling boil and maintain for five minutes. Swirl the flask and again use a glass stirring rod to remove any clay clinging to the flask walls.

11. Bring the slurry to room temperature.

VI. TITRATION WITH METHYLENE BLUE

A. Materials

1. Soil sample dispersed with TSPP solution
2. Magnetic stirrer and stirring bar
3. Methylene blue solution
4. Whatman's filter paper (hardened No. 50)
5. 1 - 50-ml beaker
6. 1 - 50-ml glass syringe with needle
7. 1 - 3-ml glass syringe with needle

B. Procedure

1. Stir the MB solution in its storage container for thirty minutes before titration (this should be done once at the start of each day of testing),

2. Transfer a portion of the MB solution to a wide mouthed glass jar,

3. Mix the slurry of soil and dispersant for thirty seconds to one minute,

4. Place a piece of filter paper atop of a beaker, such that the paper's edge is overhanging the beaker (label the paper by test number, date).

5. Titrate the MB solution by syringe. Using the 50-ml glass syringe, transfer enough MB solution to the slurry so the color of the mix turns from a greenish-blue to a dark blue,

6. Mix the slurry with the magnetic stirrer for two minutes,

7. Dip a glass stirring rod into the slurry and place a drop of the dark blue mixture onto the edge of the filter paper. Watch for a light blue halo around the circumference of the resulting dark blue dot. If a halo does not appear, add an additional 1 milliliter of MB solution to the sample using the 3-ml syringe. Mix for thirty seconds and repeat Step 7,

8. When a blue-green halo appears, do not add more MB solution. Stir the sample for another two to three minutes. Check for the halo again by placing another drop on the filter paper. If the halo does not appear, repeat Step 7, but after each 1 milliliter addition of MB solution, mix the slurry for two minutes. When the halo appears on both the initial drop and a check, the end point is reached. Record the test result as milliliters of MB solution required to reach the end point.

9. The test result (ml MB) is then entered on the bentonite calibration cure to yield a percent bentonite. This is the percent bentonite in the material passing the No. 10 sieve. The percent bentonite in the total backfill sample would then be

   (percent bentonite in minus No. 10 material)
   (percent of total sample passing No. 10 sieve).

APPENDIX C

GEOPHYSICAL METHODS FOR WINDOW DETECTION

A number of geophysical methods currently exist which potentially could be used to detect windows in soil/bentonite backfilled cutoff walls. A number of these methods were briefly investigated on a theoretical basis.

For a remote sensing investigatory method to be feasible for window detection, its signal must respond differently to a window as compared to the intact backfill. Furthermore, the signal must first reach the window and then, in its altered state, propagate to the instrument for detection. This requires that the backfill, or formation surrounding the backfill, be relatively transparent to the investigatory signal. In addition, the window of interest must exhibit parameters that are significantly different from the backfill with respect to signal propagation.

C.1 Investigation Methodology

Two general investigative approaches exist for window detection. The first, defined as "PROBE" methods, continuously detect physical parameters of the backfill in the immediate vicinity of a probe as it is inserted through the backfill. These methods have resolution similar to direct sampling of the wall, but are much faster and less expensive than actual sampling. These methods, however, will not detect windows which fall in between probe hole location. Hence, the frequency of windows would be based on statistical analyses.

The second type or "SCAN" methods, pick up changes in physical parameters as electromagnetic waves pass between tranmission/relieving stations and thus through the wall. These methods "sample" the entire wall but do not have the resolution of the probe methods. They also tend to require more sophisticated equipment and data reduction. Based on work done to date, potential candidates of each type will be treated individually below.

C.2 Probe Methods

The three primary classes of probe methods are soil borings with undisturbed sampling, down-hole geophysical methods and piezocone sounding.

C.2.1 Borings/Sampling

This method provides the most definitive evaluation of window characteristics once they have been located. This results directly from the sample retrieval capability. However, the process is very slow and costly. Past experience indicates that the wall sampling and sample testing effort needed to statistically evaluate window occurrence can easily cost up to 50 percent of the construction cost of the wall.

C.2.2 Down-Hole Geophysics

A number of individual sensing mechanisms can be incorporated in a down-hole probe such as acoustical fracture mapping, nuclear density, conductivity, etc. While some of these mechanisms may respond well to differences in windows vs. intact backfill, they all require borehole installation prior to use. As such, these methods were ruled as impractical due to the costs associated with the required boring program as above. However, these methods may become practical for window detection if packaged in a direct insertion probe.

C.2.3 Piezocone Soundings

The standard configuration of probe simultaneously detects point resistance, side friction, and excess pore pressure generation during insertion into the wall. Correlation of the three parameters indicates the type of material penetrated. The probe is inserted by a drill rig at 1 to 2 cm/sec. About 250 linear feet of probing can be done a day, and as such, the method is reasonable from a cost standpoint.

The low density and strength of the backfill in conjunction with rigid push rods result in vertical penetration. The small diameter of the probe and rods (less than 2.5 inches) renders the size of these vertical holes unimportant to the integrity of the wall. In addition, it is expected that upon extraction of the probe, the hole will close due to the plastic nature of the backfill.

C.3 Scan Methods

A number of "scan type" methods exist. The following discussion will be limited to surface methods in that cross-hole geophysics was judged too costly due to the requirement for borehole installation. Surface geophysical methods include: ground penetrating radar, various seismic techniques, acoustic monitoring, terrain conductivity, and resistivity. The following sections provide a brief outline of these methods and their probable limitations.

C.3.1 Ground Penetrating Radar (GPR)

Initial work indicated that radar would probably not be applicable in soil/bentonite cutoff wall because the backfill has a very high conductivity, thus resulting in high attenuation rates. In addition, the verticality of the wall relative to the horizontal antenna array yields poor backscattering characteristics. These observations were confirmed when GPR was used to locate the centerline of the cutoff wall at the Gilson Road site. While the wall was easily distinguishable from the native sand and gravel formation, penetration was limited to approximately 10 to 15 feet.

C.3.2 Seismic

Seismic methods were easily ruled out in that the wave velocity through intact backfill and windows is expected to be nearly identical. This is a direct result of the high influence of water on wave velocity and the fully saturated nature of all backfill materials including windows.

C.3.3 Acoustic Emission

Standard acoustic emission monitoring techniques are not appropriate for window detection because differences in intergranular shear strain are not expected in the backfill due to the presence of windows. In addition, even though seepage velocities through windows in the wall would be much higher than those through intact backfill, it is not expected that the stresses induced would generate significant emissions. Furthermore, the natural soil on either side of the wall would typically preclude piping and the associated highly turbulent flow (large emission potential).

More recent developments center around monitoring the acoustic emissions generated by probe induced material displacement during a sounding. Equipment packaging is similar to the piezoconeinstrument. This probe technique may offer potential for anomaly detection in soil/bentonite cutoff walls, but was not yet available during execution of this research.

C.3.4 Terrain Conductivity

Terrain conductivity has been shown effective in locating changes in the conductivity caused by contaminant plumes. However, a number of constraints render this method inappropriate for investigation of the occurrence of windows in the Gilson Road cutoff wall.

- This method will not respond to the physical differences between windows and the intact backfill due to the same limitations discussed under "GPR" methods and standard resistivity sounding methods.
- Cutoff walls are constructed to contain typically the most highly contaminated portion of a plume and not the entire plume. As such, contaminated water exists on both sides of the wall thus, it is difficult to locate windows via detection of contaminants escaping through wall.
- It is not likely that the detection range associated with terrain conductivity methods would allow detection of contaminated water escaping from windows located at depth, even if the water outside the wall was clean. This is particularly true for contaminant plumes which contain only organic chemicals in the parts per million concentration range.
- Windows located near the top of the wall could not be detected via differences in the conductivity of the groundwater in areas (1) where clean water exists on both sides of the wall (upgradient) and (2) where there is no gradient across the wall.

C.3.5 Resistivity

Resistivity methods using a standard electrode configurations for sounding with depth such as Wenner &

Schlumberger arrays, appear to encompass too large a mass outside the wall at trench depths approaching 100 feet. This is a direct result of the wall geometry; disproportionate depth to width distances. These geometrical limitations tend to mask the potential perturbation caused by small windows. Therefore, it is anticipated that probes would have to be inserted inside the wall and current flow between electrodes measured directly.

While this method appeared feasible, it would require significant development work and as such, was considered outside the scope of this effort.

☆U.S. GOVERNMENT PRINTING OFFICE:1987 -748-121/ 67033